

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF GRADUATE STUDIES

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

INVESTIGATING THE CAUSES OF LANDSLIDES AND REMEDIAL MEASURES FOR THE SELECTED SECTIONS ALONG ANKOBER- DULECHA ROAD UPGRADING PROJECT

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ADDIS ABABA INSTITUTE OF TECHNOLOGY

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ABSTRACT

Currently, landslide is being a serious issue in Ethiopia, especially in association with different infrastructural expansions like a road. The frequent occurrences of this event become a cause for loss of life and different resources in the vicinity of these infrastructural works. Accordingly, it needs more attention in this regard for any geotechnical structures susceptible to failure by conducting an appropriate investigation, analysis and design for these structures so that appropriate remedial solutions should be established.

This thesis presents the investigation, analysis and establishment of possible remedial measures for the three selected failed cut slope sections located at station 2+550, station 7+130 and station 12+470 of the Ankober–Dulecha road project.

In this work, first characterization of the cut slope instability areas was conducted which includes the landslide location, geological interpretation of the area and engineering classification of the failed slope soil by conducting preliminary and detailed site investigation (testing). After the characterization of the failed cut slope, the analysis was carried out employing the finite element method using 2D PLAXIS software. In the analysis, both the undrained and drained or short term and long term analyses were conducted and it was found that the drained analysis is more critical for these cut sections. Finally, using the results of the investigation and analysis, the possible remedial measures were proposed at the respective landslide locations.

It was observed that the main reasons for the failure of cut slopes are the provision of a steep cut slope, presence of groundwater at shallow depth and spring water along the slope, presence of different residual bedding profiles and low shear stress resistance of the cut slope soil. Possible remedial measures were proposed at the respective landslide locations such as provisions of gravity retaining walls coupled with appropriate control (drainage)works.

Keywords: Failed cut slope, factor of safety, PLAXIS 2D, remedial measures, drained and undrained analysis

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LIST OF ABBREVIATIONS AND SYMBOLS

AASHTO	American Association of State Highway and Transportation Officials
RHS	Right Hand Side
LHS	Left Hand Side
ERA	Ethiopian Roads Authority
FEM	Finite Element Method
LEM	Limit Equilibrium Method
FS	Factor of Safety
ASTM	American soil and Transportation Manual
SPT	Standard Penetration Test
N	SPT–blow number
BS	British Standard
UU	Unconsolidated Undrained
CU	Consolidated Undrained
CD	Consolidated Drained
Msf	Global Factor of Safety
FS_u	Factor of safety for undrained condition
FS_d	Factor of safety for undrained condition
g	Gravitational acceleration
γ_b	Bulk unit weight
c'	Cohesion in terms of effective stress
$c'_{lab.}$	Cohesion obtained from laboratory
$c'_{ans.}$	Cohesion considered as the answer
ϕ'	Angle of internal friction in terms of effective stress
$\phi_{lab.}$	Angle of internal friction obtained from laboratory
$\phi_{ans.}$	Angle of internal friction considered as the answer
E'	Effective modulus of elasticity
Ψ	Dilatancy Angle
ν'	Effective Poisson's ratio
GWT	Ground water table

1.0 INTRODUCTION

Landslide is a geo-dynamic process that comprises varieties of processes resulting in the movement of earth forming materials (soil, rock, artificial fills, and combination of them) in the downward or outward direction along with the slope formation. (Highland, 2004)

Ethiopia is currently undertaking massive infrastructural development including roads and railways, urban development, and extensive natural resources management. In this whole socio-economic development, landslides and landslide generated problems are significantly increasing from time to time due to many causative/aggravating factors.

Typically, the Ethiopian Roads Authority (ERA) has been undertaking numerous road infrastructural expansions throughout the country to address the road network distribution and accelerate the growth of the country in advance as road infrastructure is a key factor for development. However, in doing so, landslide incidents are becoming a critical issue on every project. Accordingly, these road projects which are under construction, especially traversing on an escarpment, mountainous and rolling terrains are often causing slope instability problems. This is due to the cutting of the slope as per the requirement of the road design standard. This problem is mainly experienced due to seasonal change of moisture on the slopes during the wet and dry season as the presence of water causes both the increment of shear stress in the form of seepage forces and loss of shear strength within the slope or failure surface.

During the construction of the road along with the mountainous and escarpment terrain type, massive earthwork activities are expected in the course of endeavoring to comply with the road design standard that tries to balance the terrain condition of the area. In connection to this, cut slope instability problems are the most common and source of major geotechnical difficulties encountered in such road construction projects. And also, commonly, there is a lack of commitment in road construction projects to conduct detailed investigation and design for slope instability related problems during the design stage of the projects. Consequently, the problem of instability starts when excavation works are commenced. This problem becomes a big challenge to correct and results in project cost overruns, delay in completion of the project, and financial claims among the contracting parties.

1.1 Statement of the Problem

Landslide is an old enough disaster throughout the world and its degree of risky /severity has been increasing from time to time. In Ethiopia, this event is becoming more devastating and it is commonly related to infrastructural expansion. Among other construction streams, the road construction sector is extremely sensitive to landslide occurrence, especially in mountainous and escarpment terrains. Accordingly, the country has been losing a significant amount of properties, life, natural and manmade assets due to this calamity. Thus, even if there are some endeavors with the landslide investigations, it is of paramount importance to conduct an in-depth and inclusive investigation of landslide-prone areas before any construction activities are started.

The Ankober-Dulecha Upgrading Road project is currently an ongoing project around the Amhara-Afar border. While the project has been on progress, massive landslides were experienced at different sections of the road stretch, and the contractor is forced to stop the works in these sections. Accordingly, it is found that investigating these events is mandatory to determine the causal factors which have contributed to the occurrence of these landslides and to take appropriate remedial measures.

1.2 Objective of the research

1.2.1 General objective

The ultimate aim of this research is to investigate the causes of landslides and remedial solutions by conducting detailed investigation and analysis for the selected failed slope sections of the Ankober-Dulecha road project.

1.2.2 Specific Objectives

The specific objectives of the research are;

- Characterizing and investigating the failed cut slopes
- Conducting realistic analysis for cut slopes
- Comparing drained and undrained analysis
- Proposing different remedial methods

1.3 Methodology of the Work

To achieve the objective of the study, the following methodology is employed:

- Characterization and investigation of selected failed cut slopes using field survey and laboratory tests.
- Analyses of failed slopes using PLAXIS 2D by considering both drained and undrained conditions.
- The results of the investigation and analyses are used to identify the causes and propose possible remedial measures for respective failed slope locations.

1.4 Scope of the Work

The scope of this study encompasses detailed investigation and characterization of the failed slope, analysis, and provision of optimal remedial measures at the three selected failed slopes among the total of twelve slide locations. In this thesis, the finite element analysis method is implemented, in which a 2D geometric model with plane strain condition is employed. To determine the causative factors, only static analysis of the slope is considered and in the remediation analysis, the earthquake effect is considered.

The implementation of the results of this study is limited to the selected failed slope locations. However, as most of the failed slopes along the road stretch have similar failure scenarios with the three selected failed cut slopes, the method of investigation, analysis and remedial measures proposed for these sites may be employed for other failed slope locations of this road project.

1.5 Organization of the Thesis

The thesis is organized into six main chapters. The first chapter comprises the general introduction to landslide, objective, scope, and methodology of the study. Chapter Two presents the literature review of the general slope stability concepts, general causes and remedial measures for slope instability. Chapter Three is dedicated to a brief description of the road project with due emphasis on project location, climatic conditions, geology, topography and drainage condition of the study area. Furthermore, Chapter Four highlights the investigation and analysis of the respective failed cut slopes/landslides locations. The respective remedial solutions for the slide locations are presented in Chapter Five and the conclusions and recommendations of this thesis report are discussed in Chapter Six.

2. LITERATURE REVIEW

2.1 Introduction

A landslide can be defined as the movement of the mass of debris, rock, or earth down a slope or it is a type of "mass wasting" along with the surface of rapture in the form of either curved (rotational) or planar (translational) slides.

Due to the rapid increment of the demand for engineered cut and fill slopes on construction projects, their impact on the disturbance of the balance of natural slopes has been increasing through time. Consequently, the needs of detail understanding the methods of analysis, investigation tools, and stabilization methods, to solve these slope instability problems, have been increased through time.

For the ease of understanding of the situation, dimension and geometry of landslide in the time of occurrence, it is common to represent it by graphical illustration. The most frequently used schematic representation is the one which is described in Figure 2.1. (Varnes, 1978)

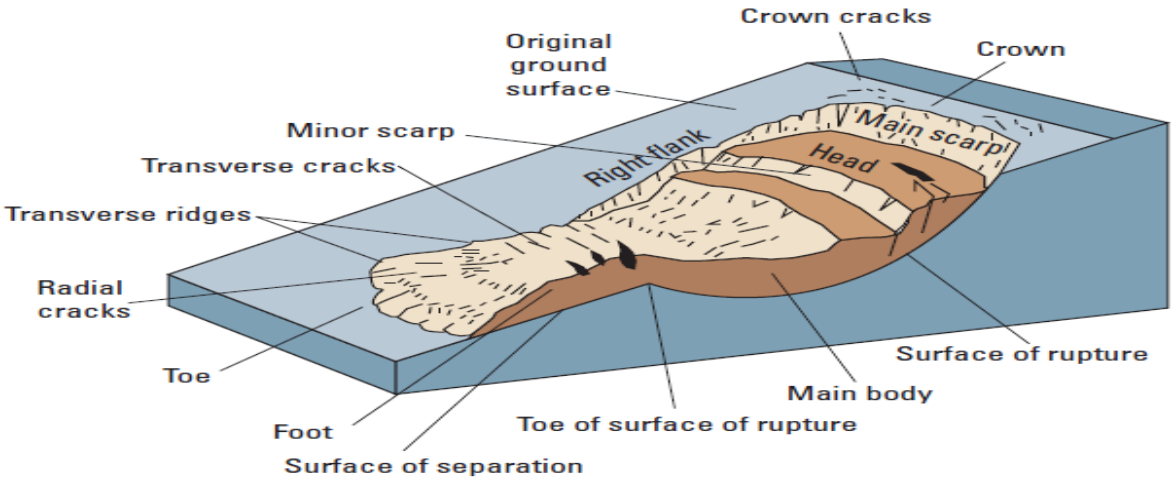


Figure 2.1 Terminologies of the landslide in the time of occurrence

2.2 Classification of Landslides

Several landslide classification methods and systems have been proposed for landslides as per the material involved in the landside and the mode of movement. Accordingly, based on the

kinematics of landslides, i.e. how movement is characterized throughout the displaced mass, the landslide is classified into the following main categories. (Varnes, 1978)

SLIDES: mostly, many types of mass movements are included in the general term of "landslide"; however, the more restrictive use of the term refers only to mass movements where there is a distinct zone of weakness that separates the slide material from the more stable underlying material. There may be rotational or translational slides (Figure 2.2 A and B).

Falls: are abrupt movements of masses of geologic materials that become detached from steep slopes or cliffs along discontinuities such as fractures, joints and bedding planes (Figure 2. D). Movement occurs by free-fall, bouncing and rolling and is strongly influenced by gravity, mechanical weathering, and the presence of interstitial water.

Topples: these are distinguished by the forward rotation of a unit or units of mostly rocks about some pivotal points which are caused by the actions of gravity, forces exerted by adjacent units, and fluids in cracks (Figure 2.2 D).

Lateral spreads: they usually occur on very gentle slopes or flat terrain. The dominant mode of movement is lateral extension accompanied by shear or tensile fractures (Figure 2.2 J). The failure is caused by liquefaction/saturated. Cohesionless sediments (usually sands and silts) are transformed from a solid into a liquefied state.

Flows: there are five basic categories of flows that differ from one another in fundamental ways.

a. Debris flows: it is a rapid mass movement in a combination of loose soil, rock, organic matter, air, and water mobilized as a slurry (Figure 2.2 F). It is caused by intense surface-water flow, due to heavy precipitation that erodes and mobilizes loose soil or rock on steep slopes.
b. Debris avalanche: This is a very rapid flow of debris (Figure 2.2 G).
c. Earth flow: a downslope movement of earth as a viscous fluid. (Figure 2.2 H).
d. Mudflow: mudflow is an earthflow consisting of material that is wet enough to flow rapidly (Figure 2.2).
e. Creep: it is a lightly slow and steady downward movement of slope-forming soil or rock (Figure 2.2 I). Movement is caused by shear stress that produces a permanent deformation.

Composite slides: combination of two or more of the above-mentioned slide types.

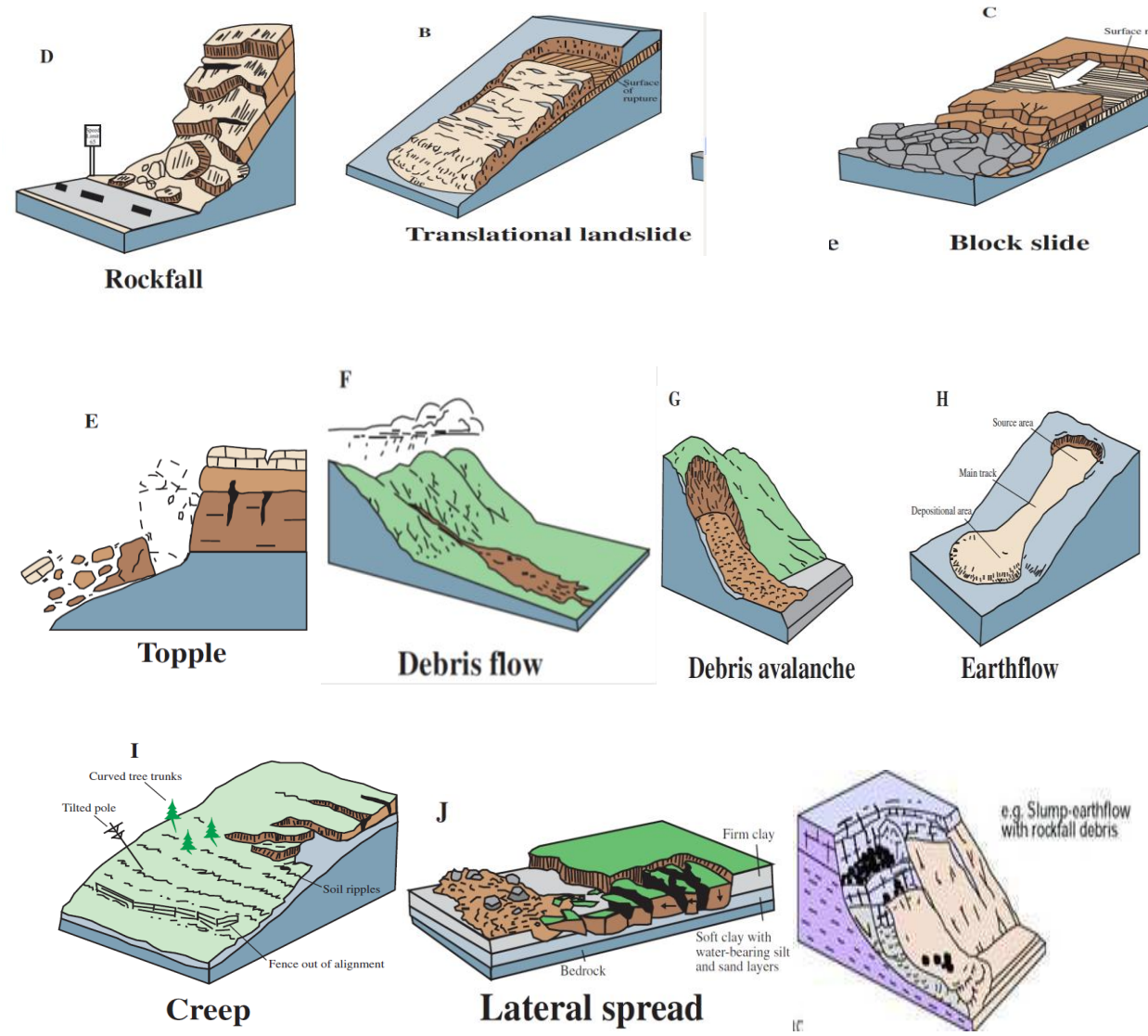


Figure 2.2 Common types of landslide representations (Varnes,1978)

Accordingly, the failure types experienced in these selected cut slope sections are mainly slid (rotational or translational) and flow type failures.

2.3 General Causes of Landslide

Slope failures are often caused by different conditions that increase shear stresses or reduce the strength capacity of the soil mass. (Abramson, et al., 1996)

Processes that are a source for the increment of the shear stresses:

- Removal of Support: erosion, natural soil movements (e.g. falls, slides, settlements), human activity (cuts and excavations, removal of retaining walls, a drawdown of bodies of water)
- Overloading: by natural causes (weight of precipitation, accumulation of materials because of past landslides), by human activity (construction of fill, buildings and other overloads at the crest)
- Large strain (steepening of riverbank), dynamic loading (pile driving), cyclic loads (train traffic), piping and earthquake.
- Removal of underlying materials that provide support (by rivers, by weathering, by underground erosion due to seepage (piping), by human activity (excavation or mining), by loss of strength of underlying material)
- Increase in lateral pressure (by water in cracks and fissures, by freezing of the water in the cracks, by an expansion of clays)

Conditions that contribute to the reduction of shear strength in the slopes soil mass are;

- Factors inherent in the nature of the materials (Composition, Structure, Secondary or inherited structure, stratifications)
- Changes caused by weathering and physiochemical activity (wetting and drying processes, hydration, removal of cementing agents)
- Effect of pore pressure
- Changes in structure (Stress release, structure degradation)

Residual soil and weathered bedrock can be weakened by preexisting discontinuities such as faults, bedding surfaces, foliations, cleavages, sheared zones, relict joints, and soil dikes. Relict joints and structures in residual soils often lose strength when saturated. Slicken sided seams or weak dikes may also preexist in residual soil and weathered rock slopes. Faults, bedding surfaces, cleavages, and foliations have more influence on rock slope stability than soil slope stability.

Slope cuts release residual horizontal stresses and cause expansion of the slope. It is essential not to undermine the toe of the cut slope but to maintain the slope as loaded as possible. The main causes of cut slope instability are overcutting the toe or over steepening the slope angle. (Abramson, et al., 1996)

Generally, a cut is the removal of support from the slope and for some slope settings which are in a slightly stable condition like colluviums or ancient landslide deposited slopes; and hence, cutting off these slopes often results in a landslide. This is due to removing the soil superimposing an artesian basin condition, interrupting a floated groundwater, causing a line of leakage on the slope face, excess steepening of the cut slope on the soft foundation causing the foundation to fail.

2.4 Prevention of Landslides

It is obvious that landslide is a highly distractive and catastrophic hazard in the world. Prevention of this risk is paramount important than any other post-treatment mechanism. Accordingly, there are many ways of landslide prevention mechanisms that should be implemented in the areas where the probability of occurrence is significantly expected. Some of the effective ways of landslide prevention mechanisms are (Asmelash, 2012; Tekalgn, 2016):

- Identifying the landslide risk through 'Landslide Hazards Mapping' as it indicates the areas where there have been moderate or high incidences and also using past experience. Consequently, implementing a plan for hazard reduction on a prioritized basis.
- Periodic inspection of facilities that are vulnerable to the landslides. This is to assess any signs of instabilities and take preventive action as much as possible at the early stage. In other words, developing long and short-term readiness strategies, communication systems, retreatment plans and resource inventories to respond to any of emergencies connected with land instability.
- Maintaining and improving the habit of land use management, drainage-system, promoting afforestation practice and reduce deforestation, gully treatment and reduction of stream/river erosion, providing retaining structures where necessary (e.g. at roadsides)
- Keeping lifeline infrastructures and other places of public access from earthquake-initiated landslides. (Derek H., 2005)
- Generally, the establishment of the agency at the national level that can do on public awareness creation and education program, facilitation of training for authorities/decision-makers and experts on landslide hazards and their mitigation mechanisms.

2.5 Possible Remedial Measures of Landslide

Stabilization of failed slopes along the failure stretch is usually difficult because of the complex geological and geomorphological nature of most slopes; however, it is mandatory to consider it as stabilization of these slopes generally reduces the driving forces and increases the resisting force and makes the slope stable.

In stabilization or remediation of slope instability, one should first understand the cause of the slope instability and the extent of remediation needed to maintain the stability of the slope for the reasonably foreseeable future conditions.

There are several methods of slope remediation methods that can be employed in slope instability related problems; however, among these, the most frequently used /recommended methods which are related to cut slope instability problems are discussed below. (Abramson, et al., 1996)

a) Drainage: the provision of drainage increases the stability of the slope whereby it reduces the effects of surface runoff and groundwater saturation in the slope by reducing the destabilizing hydrostatic, seepage, and pore pressure forces that contribute for the driving forces in the slope. Based on the site condition, it is common to provide surface drainage, subsurface drainage, or the combination of the two.

- Surface Drainage: The provision of this drainage is more effective at the head of the slide where the system of cutoff ditches that cross the headwall of the slide and the lateral drains to lead runoff water around the edge of the slide to be effective. It needs an appropriate gradient to ensure the rapid flows are away from the unstable area. This drainage also includes concrete interceptors drains along the crest of the cut section, along with benches and along with the toes to capture all freely drained surface runoff water. Most of the time, this type of drainage is economical for shallow soils deposits that overlaying bedrock or hard impermeable tills.
- Subsurface Drainage: These drainages are associated with the installation of horizontal and vertical drainage walls and drainage tunnels. This type of drainage mostly applicable when the investigation has revealed the presence of groundwater, slide movement and positive subsurface drain; otherwise, they are costly and difficult to implement when compared with other alternatives. The drainage-related expenses are generally less when

these measures are incorporated in the preliminary design process than when they are included as remedial measures during construction.

b) Unloading: this method is used for the reduction of the driving forces within the failed mass. This is accomplished by removing the entire or part of the failed mass. Generally, it is only practicable for small slumps and rotational failure. The large-scale exaction of massive failures is usually not recommended as they may further trigger larger slides and destabilize the ground upslope in which in the time of undercutting, the slope will be weakened. In advance, this method includes removing all unstable or potentially unstable material, flattening the slopes and benching.

- Removal of Unstable Material: removal of the soil from the head of failed slope reduces the driving forces and thereby improves stability; this method is more effective for cuts in the deep soil where rotational slides are expected. Due to the removal of the failed mass and the reduction of the height of the slope, the slopes become flatter. On the other hand, reducing the height of the slope leads to the reduction of the driving force in the failure plane.
- Benches: benches are a series of steps cut into a deep soil or rock face for the purpose in reducing the incidence of shallow failures; but, generally are not very efficient in improving the overall slope stability for which other methods are recommended to be more confidential. In other words, benches are useful in providing protection structures, control of surface drainages, provide working areas and others.
- Flattening: this is the reduction of slope angle, the weight of the material, and the possibility of stream/river undercutting, construction loading, or other slope modifications techniques.
- Backfilling with Lightweight Material: it is related to weight reduction and it can be achieved through excavating the upper soil of the slope and replacing it with a lightweight backfill material and then, covering it with a thin layer of coarse aggregate.

c) Strengthening the Slopes: this method is associated with the increment of the forces that can resist the movement of the slopes. This may include (CWCE, 2018):

- Rock-fill Buttress: a simple method to improve slope stability by increasing the weight of the material at the toe and that result for the creation of a counterforce that can resist failure. This is achieved by dumping of berms/buttress of earth-fill and broken rock or riprap onto the toe/base of the slope.
- Stream Channel Lining: stabilization of stream and sides of stream channel by lining the channel with high-quality concrete or preferably reinforcing by steel fiber mat for the resistance of abrasion.
- Check Dam: check dams are a small, sediment storage barrier built across the channels of steep gullies to stabilize the channel bed. They are commonly used to control channelized debris flow, raveling and shallow slides in the mountain, to mitigate the incidence of failure by reducing channel gradient and by providing toe support to the gully slopes.

d) Retaining Wall Structures: rigid structures that are constructed at the base and toe of the sliding surface. The most frequently used retaining structures for road construction associated with mass failures/slides are;

- Gravity Retaining Wall: is employed to bear the soil mass or backfill on the backside of the wall depending on their masses or self-weights to counterbalance the forces coming from retained soil mass.
- Reinforced Retaining Wall: it is generally used to retain earth or other material to maintain unequal levels on two faces. This type of retaining wall resists the lateral pressure by structural action such as bending and results in a thinner section.
- Mechanically Stabilized Earth (MSE): it is a patented system for the construction of fills at a very steep to vertical angles without using supporting structures at the face the fill. This system uses a horizontal layer of flexible metal strips (geogrid) within the fill to form a composite earth-metal system with high strength. This system increases the shear strength and bearing capacity of the backfill by intercepting the slip surface of the slopes.
- Soil Nail Wall: it is a structural element that transfers the loads to the ground in excavation reinforcement applications. In the process of soil nailing excavation walls and slopes are stabilized in situ by installing relatively short, fully bonded steel bars or other reinforcing materials according to a regular and relatively closely spaced pattern. It has been used in a

variety of civil engineering projects as support of deep excavation as well as slope stabilization like highway cut slope construction or temporary building excavation support. (Rowe, 2001)

Based on the nail installation techniques, there are various types of soil nail walls; drilled and grouted soil nails, driven soil nails, hollow bar soil nails, jet-grouted soil nails and launched soil nails. (Xiao, 2015)

- Pile Wall: A structural element executed in the ground to transfer vertical and lateral loads to the deep foundation soil. There are different types of pile walls. Contiguous (intermittent) bored piles are piles with spacing between the piles is greater than the diameter of piles and the common diameters of these pile are 0.60, 0.80, 1.00m. Tangent piles with grouting in between can be used when groundwater exists. Secant bored pile walls are formed by keeping the spacing of piles less than the diameter. It is a watertight wall. (Venkata Ramasubbarao Godavarthi, 2011)

Mostly, retaining structures are required in order to give supports for the cut or fill slopes when cuts or fills are required and if there is insufficient space or right of way restriction along the slope. The walls should be deep enough so that the critical slip surface passes around it and the slope stability is ensured.

e) Avoidance: realigning of road stretches that are affected by the slide when a better alternative route is available. This method is cost-effective if the realigned section of the road is short.

f) Plantation: if not controlled, surface erosion and small with shallow slope failures can lead to larger problems that cannot be controlled. Planting grasses reduces surface erosion and planting with shrubs adds the vegetative cover, which in turn will enhance the slope stability. After the execution of benching, reducing slope angle, removing cut-back overhangs and drainage provisions, it is advisable to implement plantation as it increases the effectiveness of these remedial techniques. (CWCE, 2018)

2.6 Methods of Slope Stability Analysis

Slope stability analysis is one of the major tasks in the field of geotechnical engineering for a long time ago. The problems in the slope stability analysis are statically indeterminate and to manage

this difficulty, different methods of analysis are devised through time. Accordingly, the two well–known approaches in this regard are the Limit Equilibrium method and the Finite Element Method.

2.6.1 Limit Equilibrium Method (LEM)

It is a conventional slope stability analysis method that has been widely used for many years in the landslide problems. This method is mostly based on the techniques of slices in which a failure surface is assumed and the slide mass along with the failure surface is divide into a smaller number of slices. These slices may be cut in vertical, horizontal and inclined orientations; however, the vertical cut is preferred by most investigators. It is carried out by considering a single factor of safety throughout the whole failure mass, employing assumptions to reduce the problem of indeterminacy and calculating the factor of safety from the force and/or moment equilibrium equations. LEM has its own advantages and disadvantages in geotechnical engineering problems.

Advantages;

- The relative simplicity and hence, geotechnical engineers have experienced it well over the years for the calculation of factor of safety
- Its ability to evaluate the sensitivity of stability for the various input parameters

Disadvantages;

- Consideration of assumptions for the form or location of failure surface, side forces of the slice and their direction; and this may lead to unnecessary error or misconception.
- The assumption of constant FS along the entire slip surface is an oversimplification, especially if different soil materials exist along the failure surface.
- The absence of stress–strain equilibrium in the soil, that is, stress or deformation change is not considered.

This method can further be divided into two based on the reliability of the input data. These are;

- Deterministic slope stability analysis: in this method, the FS is computed based on a fixed set of conditions and material parameters. If the factor of FS is less than unity, the slope considered being unstable or susceptible to failure. A deterministic analysis suffers from

limitations such as that the variability of the input parameters is not considered. (Duncan, 1996)

- Probabilistic slope stability analysis: this method allows for the consideration of variability in the input parameters that quantifies the probability of failure of a slope. The variability of material can be considered. (Duncan, 1996)

2.6.2 Finite Element Method (FEM)

In this method, the slope is divided into a finite number of regions or elements using appropriate constitutive laws for the materials in the slope and these elements are interconnected at their nodes and predefined boundaries of the continuum. Due to the readily availability of personal computers, the Finite Element method has been increasingly used in slope stability analysis. This method is more general and bypasses many of the drawbacks of LEM as stated below;

- Employed in the condition when the soil profile, configuration and depositions are becoming more complex.
- It is more accurate than the limit equilibrium method as it does not require any prior assumptions for the shape and the location of the critical slip surface.
- This method computes the stability factor of the slope using the stress-strain behavior of the soil obtained from a finite element stress analysis that removes the assumption applied in LEM to change a static-indeterminate problem to a statically determinate one.
- It can be used to calculate stresses, movements/displacement, pore pressure and other characteristics of earth mass during construction. And also used to calculate seepage failure and monitor progressive failure. (Duncan, 1996)
- As the slice technique is not implemented in the finite element approach, there is no assumption about slice side forces.

Although this method of analysis is more appropriate, it is quite sensitive to the quality of the input data and the detail of the model setup. In connection to this, FEM of slope stability analysis is conducted for academic purposes than practical application. This is further due to; the lake of well-documented data for the sensitivities of the employed soil material properties, slope configuration, groundwater table, effect of mesh size, different element types and element nodes in the numerical simulation. (Lu and Chih-Wei., 2011)

Typically, FEM has used the displacement type of formulation for the quantification of field variables (i.e., displacement, stress and strains) which are measured at the nodes and ensures continuities along the grid lines using suitable interpolation relations.

In the implementation of FEM for any geotechnical problems, the following should be considered;

- The selection of an appropriate constitutive model which can fit the encountered problem
- Considering the presence of different types of finite elements (e.g. triangular, quadrilateral, or isoparametric).
- Ensure the availability of laboratory and field test data that are required to define the soil property and hence the soil model.
- Selection of suitable types of software packages (PLAXIS, ABAQUS).
- Setting out an appropriate geometric model (2D or 3D) depending on the actual condition of the problem encountered.
- Considering drainage condition (undrained and drained) for the subject geotechnical structure.

For the sake of thorough understanding and to ensure the practical applicability, FEM is used /implemented in this thesis for the computation of factor of safety and stability analysis of the subject failed slopes.

2.6.2.1 PLAXIS Software

It is a finite element program or a simulation tool for many finite element based problems, specifically used for stability and deformation analysis in geotechnical applications. The package uses a useful graphical user interface that enables users to rapidly generate a geometry model and finite element meshes. Since the PLAXIS code and the corresponding soil models are developed to perform a calculation of any of the realistic physical problems, it enables the user to choose different soil models based on the mechanical deformation behavior of soil to be simulated. It is equipped with features to deal with various aspects of complex geotechnical structures and construction processes using robust and theoretically sound computational procedures. (Brinkgreve, et al., 2002)

Dimension in the analysis is an important concept that can significantly affect the result of calculation in this software package. Accordingly, PLAXIS software may be either 2D or 3D based on the physical constraint of the problems which are required to be analyzed. And hence, by considering the complexity of model geometry and diversity of material properties, certain differences are expected between 2D and 3D analysis. (Habibnezhad, 2014)

PLAXIS-2D (Plane Strain):

- It is more of conservative and considerably simplifies the actual physical problems.
- It is more practical for homogenous soil types, failure surface and failure mechanism.
- In 2D modeling, 3D geotechnical structures are assumed as they have an infinite width in the Z direction and their 3-dimensional behavior of failure surface is easily neglected and hence it is called plane strain condition.
- Due to its simplicity and optimization of time of calculation without affecting the stability negatively, 2D analysis is more appropriate for slope design. (Duncan, 1992)

PLAXIS-3D:

As failure modes of almost all of any geotechnical structures are in 3D form, the 3D analysis can result in an accurate output. Assumption is not considered to create and analyze 3-dimensional models and it reflects the real behavior of slope.

Further, this form of modeling and analysis is recommended for the condition mentioned herein below. (Auvinet and Gonzalez, 2000)

- For short width geotechnical structures like earth dams built-in a narrow valley.
- For soils subjected to concentrated loading.
- For the soil properties that vary significantly along the longitudinal direction of the slope.
- For complex geometry and irregular failure surface.

2.7 Constitutive Model

The Constitutive/material model is a set of a mathematical equation that describes the relationship between stress and strain of materials. Different types of constitutive models have been developed by several scholars to explain various aspects of soil behavior. These commonly known constitutive models that describe the stress-strain relationship of the soils are the Elastic Model,

Mohr-Coulomb Model (MC), Harding soil model (isotropic hardening), Soft soil – creep model (time-dependent behavior), and Soft-Soil model and modified cam-clay model. Among these models, the Mohr-Coulomb Model is employed in this thesis, due to;

- It is relatively quick and simple
- Its procedure reduces error and
- It is appropriate for most soils

Mohr-Coulomb Model is an elastic- perfectly plastic model implies the behavior of the soil is linear elastic up to a certain stress limit and after while the soil behaves perfectly plastic in which the strain is irreversible after a stress decrease. This model requires five input parameters for the soil; Young's modulus (E), Poisson's ratio (ν), Friction angle (Φ), Cohesion (C) and Dilatancy angle (Ψ). The value of the stiffness parameter (E) and cohesion (C) is in general chosen as a representative value that is consistent with the stress level in the soil. (Brinkgreve, et al., 2002)

2.8 Geotechnical Parameters

The basic Geotechnical parameters which are employed in PLAXIS software under the Mohr-Coulomb model are;

2.8.1 Stiffness Modulus (E_s)

It is an elastic soil parameter that describes the load-settlement behavior of soils and governs the results of settlement-related problems. It is the ratio of the stress along an axis over the strain along that axis in the range of elastic soil behavior. To simulate the actual condition of the soil using the finite element method, it is highly recommended to use practical and reasonable stiffness values that represent the in situ conditions.

There are several methods available to estimate the soil stiffness modulus which includes;

- Test methods: unconfined compression tests, triaxial compression tests and in situ tests.
- Empirical relations and range values
- Using code provisions: German code

However, for this study, the German code provision is employed, because it considers penetration resistance of the soil, confining pressure and soil types and hence, the provision of this code is given by equation 2.1. (DIN 4094-3, 2002)

$$E_s = \mu p_a \left(\frac{\sigma_z + 0.5\Delta\sigma_z}{p_a}\right)^w \quad (2.1)$$

Where:

μ = stiffness coefficient depending on values of N-SPT

w = stiffness exponent, which has a value of 0.5 for non-cohesive soil & 0.6 for cohesive soils

σ_z = overburden pressure at a depth z below the foundation level

$\Delta\sigma_z$ = additional vertical stress due to the loads from the superstructure at a depth z

Pa = average atmospheric pressure, taken as 101.4 KN/m^2.

And to account for the effective modulus elasticity for PLAXIS analysis, it is common to employ equation 2.2. (Brinkgreve, et al., 2002)

$$E' = \frac{2(1+\nu)E_s}{3} \quad (2.2)$$

2.8.2. Cohesion (c)

Soil cohesion is the force that holds together molecules or like particles within a soil. It is usually determined in the laboratory from the direct shear test and triaxial shear test. Unconfined Compressive Strength (S_{uc}) can be determined in the laboratory using the triaxial test or the unconfined compressive strength test. There are also correlations for S_{uc} with shear strength as estimated from the field using vane shear tests. For this study, the cohesion of the soil has already been determined from the triaxial shear test and SPT N-value.

2.8.3. Friction Angle (ϕ)

A friction angle is a measure of the ability of a unit of soils or rock to withstand shear stress. It can be determined in the laboratory by the direct shear test or the triaxial shear test. There are also Correlations to quantify the friction angle of granular soils. For this study, the friction angle of the soil has been determined from the triaxial shear test and SPT N-value.

2.8.4. Unit Weight (γ)

The unit weight of a soil mass is the ratio of the total weight of the soil to the total volume of the soil. Unit weight is usually determined in the laboratory by measuring the weight and volume of a relatively undisturbed soil sample obtained from the field. Measuring the unit weight of soil directly in the field might be done by sand cone test, rubber balloon, or nuclear densitometer. For this study, the laboratory test result was taken.

2.8.5 Poisson's Ratio (ν)

Poisson's ratio is a property that describes the volume change of material in a direction perpendicular to the application of a load. It is defined as the ratio of the axial compression to the lateral expansion of soils. It is recommended a range of values of Poison's ratio between 0.4 and 0.5 for most clay and 0.2 to 0.4 for medium to dense cohesionless soil (Bowles, 1996). However, a typical value of 0.35 was taken for the numerical analysis of this study as the PLAXIS (8.2 versions) did not accept a value greater than this one. (Brinkgreve, et al., 2002)

2.8.6 Dilatancy Angle (Ψ)

Apart from heavily over-consolidated layers, clay soil tends to show little dilatancy. Dilatancy of sand soil depends on both the density and friction angle. For quartz sands, the order of magnitude is given by $\Psi=\Phi-30$. However, for the Φ values less than 30°, dilatancy values are mostly zero. A small negative value is only realistic for extremely loose sands. (Brinkgreve, et al., 2002)

2.9 Drainage Conditions

Depending on the duration of loading and permeability, a distinction should be made between the drained and undrained conditions of the soil for the proper consideration of whether total or effective stress analysis is applicable in the course of analysis. (Getnet, 2016)

2.9.1 Drained and Undrained Condition

The main point here is whether or not changes in load cause change in pore pressure. The difference in these conditions is time.

Undrained condition implies a condition where changes in loads occur more rapidly than water can flow in or out of the soil or undrained condition refers to the condition where drainage is restricted. The pore pressures increase or decrease in response to the changes in loads.

Drained condition refers to a condition where changes in load are slow enough, or remain in place long enough, so that water can flow in or out of the soil, permitting the soil to reach a state of equilibrium concerning water flow. Shortly, the drained condition refers to the condition where drainage is allowed. The pore pressures in the drained conditions are controlled by the hydraulic boundary conditions and are unaffected by the changes in loads.

As cohesive materials or clays generally possess less permeability compared to sand, thus, the movement of water is restricted whenever there is a change in volume. So, for clay soils, it needs years to dissipate the excess pore water pressure before the effective equilibrium is reached. Shortly, drained condition denotes the condition where drainage is allowed, while undrained condition refers to the condition where drainage is restricted. Besides, the drained and undrained condition of cohesive soils, it should be noted that there is a decline in the strength of cohesive soils from its peak strength to its residual strength due to restructuring. (Getnet, 2016)

2.9.2 Effective and Total Stress Analyses

For effective stress analyses, the pore pressures along the shear surfaces are subtracted from the total stresses to determine effective normal stresses, which are used to evaluate shear strengths. Therefore, to perform effective stress analyses, it is necessary to estimate the pore pressures at every point along the shear surface. These pore pressures can be evaluated with relatively good accuracy for drained conditions, where their values are determined by the response of the soil to external loads. (Getnet, 2016)

In total stress analyses, pore pressures are not subtracted from the total stresses, because shear strengths are related to total stresses. Therefore, it is not necessary to evaluate and subtract pore pressures to perform total stress analyses. The total stress analyses apply only to undrained conditions. The basic premise of total stress analysis is that the pore pressure due to undrained loading is determined by the behavior of the soil. For a given value of total stress on the potential failure plane, there is a unique value of pore pressure and therefore a unique value of effective

stress. Thus, although shear strength is indeed controlled by effective stress, the undrained condition can relate to the undrained condition. This line of reasoning does not apply to drained conditions, where pore pressures are controlled by hydraulic boundary conditions rather than the response of the soil to external loads. (Duncan and G.Wrigth, 2000)

2.9.3 Undrained and Drained Shear Strengths

Measured soil strength parameters are not unique. Soil type, strain history, and loading conditions (time) are the major factors to consider in the selection of strength parameters. Here, the strain history controls whether peak, ultimate, or residual strength will prevail, while the time element refers to short term or undrained conditions and long term or drained conditions.

Undrained Strength is the shear strength of the soil when loaded to failure under undrained conditions. In the field, closely approximating undrained conditions result when loads are applied to a mass of soil faster than the soil can drain. In the laboratory, undrained conditions are achieved by loading test specimens so rapidly that they cannot drain, or by sealing them in impermeable membranes. Here, to avoid the high strain rates that are not representative of the field conditions, one needs to control drainage through the use of impermeable membrane rather than very high rates of loading.

Drained Strength is the shear strength of the soil when it is loaded slowly enough so that no excess pore pressures are induced by applied loads. In the field, drained conditions result when loads are applied slowly to a mass of soil, or where they persist for a long enough time so that the soil can drain. In the laboratory, drained conditions are achieved by loading test specimens slowly so that excess pore pressures do not develop as the soil is loaded. (Duncan and G.Wrigth, 2000; Getnet, 2016)

2.9.4 Analyses of Drained and Undrained Conditions

Short-Term Analyses:

Depending on the permeability of the soil, short term analysis can be conducted during or following the construction using either drained or undrained strengths.

Cohesive soils like clay are significantly impermeable as compared to sand and hence, the movement of water is restricted for this short duration whenever there is a change in volume. For these cohesive soils, undrained shear strength is employed, and is carried out using total unit weights and total shear strength parameters; however, in PLAXIS 8.2 version, effective shear strength parameters shall be employed. (Getnet, 2016)

For sand soils that can drain freely; however, drained strengths are used; shear strengths are expressed in terms of effective stresses, and pore pressures are defined based on either water table information or appropriate seepage analysis. Undrained strengths for some soils and drained strengths for others can be used in the same analysis. (Getnet, 2016)

Long – Term Analyses:

Overtime after construction the soil in the slopes may either swell due to the increment of water content or consolidate as of the decrement of water content. Long – term stability analyses are performed to reflect the conditions after these changes have occurred. Shear strengths are expressed in terms of effective stresses and the pore water pressures are estimated from the most adverse groundwater conditions anticipated during the life of the slope. (Duncan and G.Wrigth, 2000; Getnet, 2016)

2.10 Factor of Safety

In slope stability analysis, the factor of safety is usually defined as a ratio of the available shear strength to that of the mobilized shear stress at developing failure. It is a primary index for the determination of the closeness or farness of the slope from failure condition. There are several ways to formulate factor of safety, FS, which includes;

- F_r/F_d Resisting Force / Driving Force
- M_r/M_d Resisting Moment / Driving Moment
- H_c / H Critical Height / Slope Height
- S/τ Available shear stress/shear stress at equilibrium

Among these, the last definition is widely used. (Meng, 2006)

Various codes and guidelines are recommended for the minimum factor of safety to be from 1 to 1.5 as shown in Table 2.1.

Table 2. 1 Different recommended values of minimum FS for different conditions of the slope

No.	Condition of the slope	Factor of safety	Sources
1	For the first time slide	1.3 to 1.4	(BS 6031., 2009)
	For a slide with a pre-existing slip surface	1.2	
2	For permanent loading condition	1.5	(NAVFAC DM 7.1, 1986)
	For transient load such as earthquake	1.15 to 1.2	
3	For unreinforced slope and embankment on soft ground	1.2	(JKR Road Work, 2010)
	For reinforced slope	1.5	
4	For new slope depending on the risk	1.0 to 1.4	(Geoguide, 1984)
	For existing slope depending on the risk	1.0 to 1.2	
	If no risk of danger to person/property for peak strength	1.25	
5	For static roadside slope stability	1.3	(Ethiopian Roads Authority, 2013)

The factor of safety in PLAXIS 2D is computed using the ϕ-c reduction concept. In this type of calculation, the load advancement procedure is followed. The incremental multiplier 'Msf' is used to specify the increment of the strength reduction of the previous calculation step. The strength parameters are reduced successively in each step until all the steps have been performed. The final step should result in a fully developed failure mechanism, if not the calculation must be repeated with a larger number of additional steps. Once the failure mechanism is reached, the factor of safety is given in equation 2.3.

$$FS = \frac{\text{Available shear strength}}{\text{shear strength at failure}} = \Sigma Msf \qquad (2.3)$$

This is a global factor of safety at failure. (Brinkgreve, et al., 2002)

2.11 Previous Studies

Due to its complex geomorphological, hydrological and geological setups, the rift margins and highlands of Ethiopia have been frequently affected by new and reactivated landslides. Even though there are several slope instability problems in different parts of the country, only a few endeavors are made to the potential causes and avert these challenges. Even these of a few so far reported landslides were recognized either because of they are being along major economic routes especially along with infrastructural expansion areas (road, railway, dams) or in some extent when the people on the vicinity of the landslides are affected and they need critical support from the government. Otherwise, the issue of landslides and landslide-generated ground failures remain far above what is actually reported. The degree of landslide disruption has been increasing through time and is affecting infrastructures, agricultural lands and the natural balance of the land. (Asmelash, 2012)

Studies regarding landslide in this specific study area (Ankober area) are rare; however, there are some studies along the rift margins, in general, around Debresina in particular concerning geomorphology, hydrological and seismicity. Ankober is located along the rift margins like that of Debresina (around 70km far from Ankober) and these two locations have similar geomorphological, hydrological and geological setups. Starting from this concept, it is possible to assess the previous studies of the area.

The western Afar rift margin has an estimated length of about 550km from the Ankober area (southern end) to the Dallol area (its northern end). And these Rift margins are one of the most potential regions for geo-hazards which includes landslide, seismic, and land degradation. This part of the rift is densely populated and contains several towns, infrastructures such as the roads, large bridges, road and railway tunnels, dams/reservoirs and other investments. In general, in this region (western bounding of rift margin) connects the three important structures, namely the NW-SE trending structures (parallel to the general trend of the red sea); NE - SW trending structures (parallel to the main Ethiopian rift) and the E-W trending (parallel to the Gulf of Aden). The southwestern bounding rift margin, where the study area is found is characterized by these three important regional structures controlling the deep-seated landslides along the rift margins.

And also, Due to the presence of steep slope and intense faulting and fracturing, these areas are characterized by high weathering, erosion and slope instabilities. (Asmelash, 2012)

Although there is no clearly documented evidence when and how different scales of landslides happened around the rift margin, some local information revealed that they are triggered either by the heavy rainfall or with the earthquakes.

Some scholars summarized many of the favorable conditions for landslides around highlands and the rift margins of Ethiopia which are mainly connected with their regional characteristics and include: (Woldearegay, 2005; Ayele, et al., 2009; Asmelash, 2012)

- The region connects different fault systems (NE–SW, NW–SE and E–W) which are the major controlling factor for the large and deep–seated landslide along this rift margin. The E–W marginal fault systems are also controlling the drainage system of the margin.
- Formed by series of regional step that forms normal faults and these serve as a path for regional groundwater flow that enhances slope instability; can trigger local earthquakes; facilitates high differential weathering and deteriorates geotechnical properties of the surrounding material.
- As the area is found at the border of the seismo–tectonically active Afar rift valley, it can be easily reactivated by the volcanic eruption, diking and associated earthquakes. The earthquakes range from small to medium level in this rift margin, where the Ankober area is situated.
- The presence of high rainfall intensity and steep slopes around the top of the rift margin and enhances high runoff and erosion favoring slope instability in the area.
- The area is highly dismembered by the drainage systems following the faults and lineaments and hence characterized by deep gorges.
- Characterized by the fragile lithology (paleosols and lacustrine deposits, agglomerates, ignimbrites, various tuffs) as intercalation at the volcanic rocks as well as the presence of considerable thickness of colluvium soil and fault deposit at the foot of the steep slopes also highly influencing the stability condition following the regional trend of the rift margin.
- Features that act as drainage structures between highlands and the lowlands and several springs are emerging either following the hydraulic differences of the various lithological

stratifications (contact springs) or following the geological structures (fracture springs). The high fluctuation of their discharge between the dry season and the wet season enhances further slope instability.

- The region is a densely populated area and their impact on the local environment degradation is considerably high. There are intense agricultural activities and deforestations are common and are daily activities in the steep slopes which are aggravating the slope instability.
- Improper slope cuts during the construction of road infrastructures that connect the concentrated towns found along with these areas

3. GENERAL OVERVIEW AND REGIONAL SET-UP OF THE STUDY AREA

3.1. Background Information on the Road Project

The Ethiopian government represented by the Ethiopian Roads Authority (ERA) has signed a contract agreement with the local contractor, Sunshine Construction PLC, for the construction works of the Ankober-Dulecha-Awash Arba, Lot 1: Ankober-Dulecha Road Project (~40km). The road project starts at the outskirts of the historically known place, Ankober town, on the highland section of Ethiopia which has an elevation of 2726m above mean sea level and it descends to the southeast direction of the lowland section and end of the project, Dulecha town, with an elevation of 1060 above means sea level.

The scope of the contract was the construction of a 40 km road to an asphalt concrete (AC) standard with an initial project cost of ETB 858,761,475.70 with a contract period of 900 calendar days, having a commencement date of March 17, 2016. (C-Tech Engineering Plc., 2018)

3.2. Location and Accessibility to the Road Project

The Ankober- Dulecha road project (study area) is located in the central-eastern part of the country and connects the two regions, Amhara and Afar Regional States, along Ankober-Awash Arba –Djibouti road corridor. The road project is the middle lot of the Debreberhan-Awash Arba Junction road project stretching between Ankober (Amhara region) and Dulecha (Afar region) towns.

As shown in Figure 3.1, the Debreberhan-Awash Arba road stretch starts from Debreberhan town along the Addis-Desse- Mekele trunk road which is about 130km northeast of the capital, Addis Ababa, and traverses in the easterly direction to the end of the project (Awash Arba junction) by passing through several towns and rural villages with a total estimated length of 135Km. Out of the total length of the stretch, Ankober- Dulecha road project covers about 40km length.

This road will be used as an alternative route for the Addis-Adama-Awash-Djibouti import and export road corridor through Addis-Debreberhan-Awash Arba-Djibouti and also facilitates the eastern to the northern journey of the country without crossing Addis Ababa.

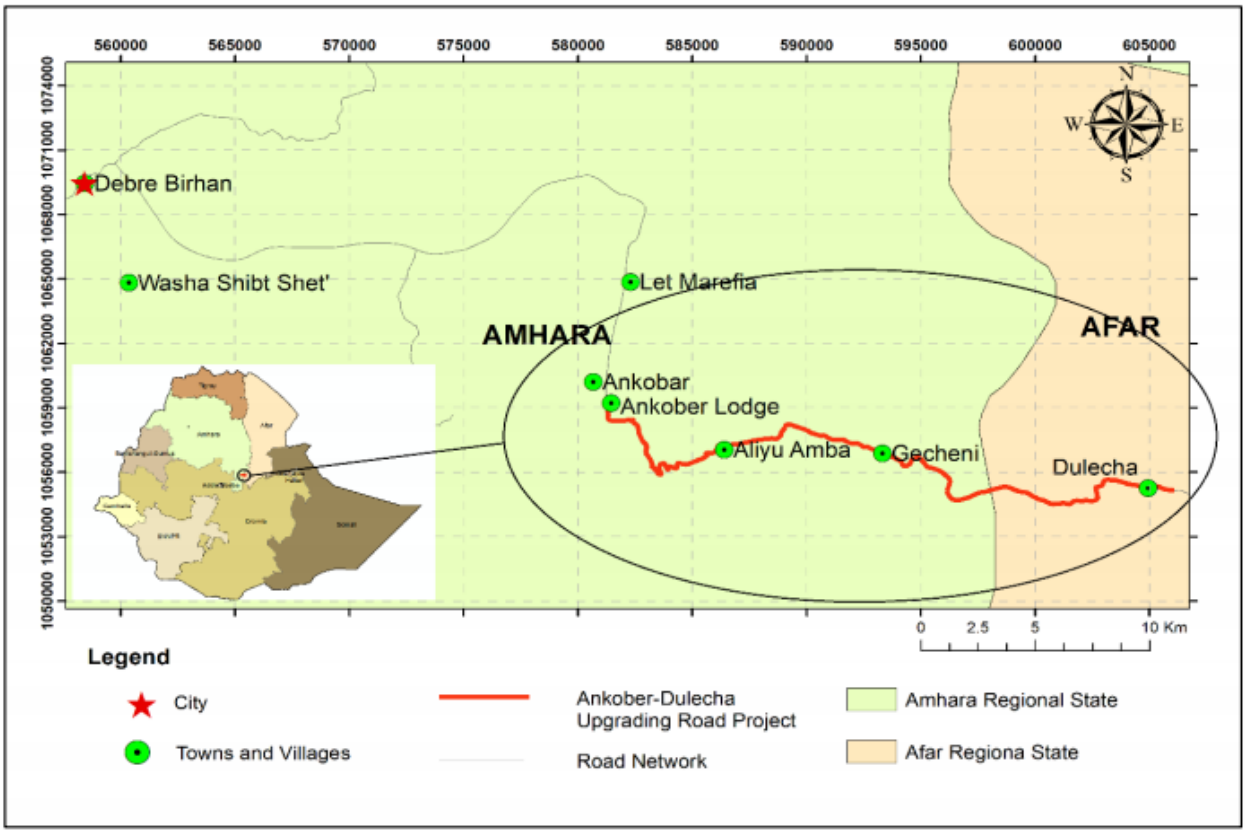


Figure 3. 1: Location map of the study area (Zerihun, et al., 2020)

3.3. Land Use and Vegetation Cover

The land usage along the road alignment is mostly cultivated land and some bare land at the end of the road project. The main livelihood of almost all the nearby people of this project is dependent on mixed farming except at the end of the project that is mostly dependent on cattle breeding.

The dominant vegetation type of the area includes eucalyptus trees, junipers, small shrubs and many undifferentiated evergreen plants (Figure 3.2). Dense eucalyptus trees are found in highly elevated areas of the road alignment. Acacia and scrubs are dominantly found along stream, ridges and around the end of the road project, mostly at the low lands.



Figure 3. 2: Land use and vegetation of the study area

3.4 Topography and Drainage Condition of the Project Area

The topography of the road corridor varies from mountainous and escarpment to flat terrain within a short distance. Particularly, at the beginning of the road project, the topography has a very steep and rugged type of nature. The altitude range from the highest point at Ankober, 2970m, down to the Dulecha, 1060m. More than 40% of the project length comprises mountainous and escarpment terrain type and due to this condition, the area is highly exposed to landslide hazards especially during the time of cutting of the natural slope. (C–Tech Engineering Plc., 2018)

The project road almost follows the old alignment (existing gravel road) except at station 5+400, where it branches off to the east in an attempt to avoid a 5.5km steep and winding portion of the

old road section. This new alignment then stretches for a total of 8.5km to rejoin the old alignment which has an extra of about 3.0km length than the old road. This new alignment traverses a geologically and tectonically more complex territory that is prone to land-sliding (Figure 3.3).

The area is a part of the Awash basin. It covers along major rivers including Melka Jebdu, Gayseno and Awash. Most of the rivers and streams in the road corridor are young streams with dendritic nature characterized by flash floods due to steep topography that flow towards the Awash River.

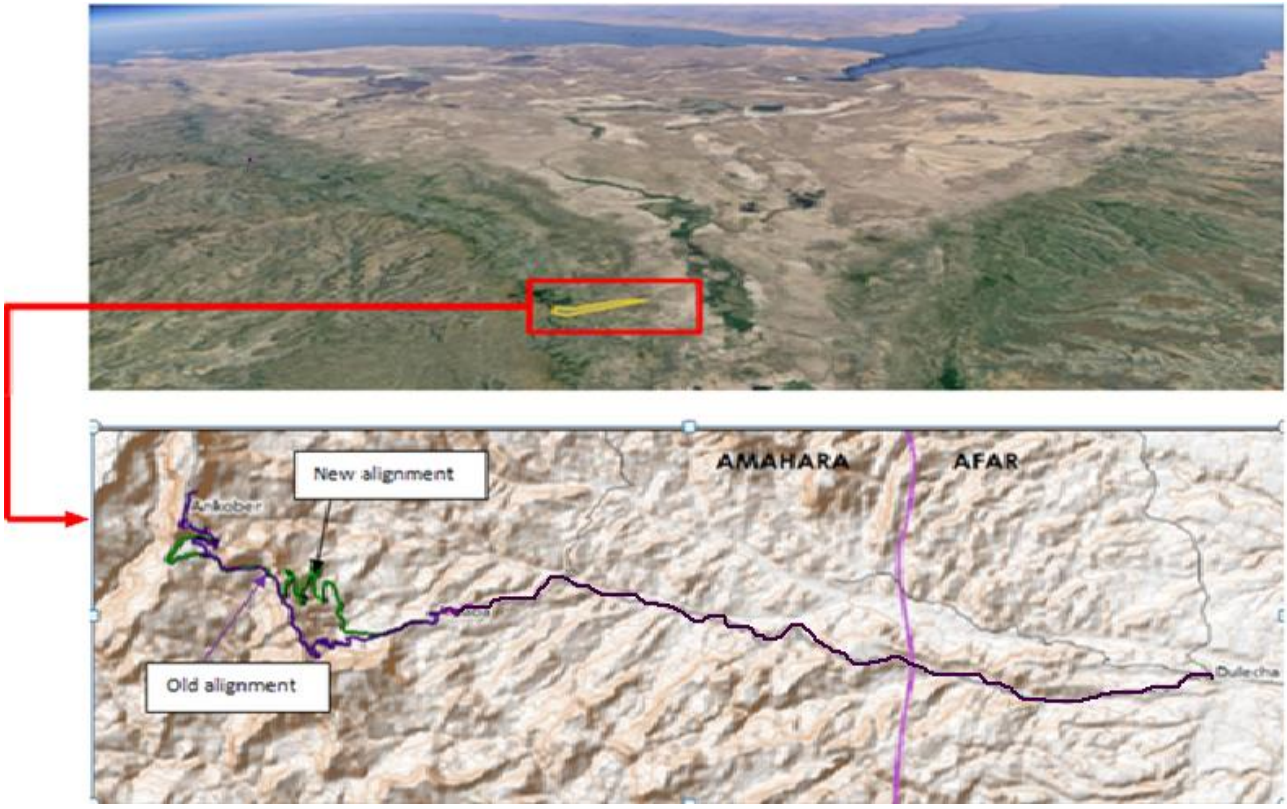


Figure 3. 3: Digital elevation model of Ankober-Dulecha road project

3.5. Geomorphology and Geology of the Study Area

3.5.1 Geomorphology

The geomorphic setting of the region along the route corridor is mainly controlled by the internal (tectonic and volcanic) and external force activities that took place in the past geological events.

The Ankober town and its surrounding areas are part of the central Ethiopia plateau and the southwestern part of the rift margin. Based on morphology, the area has physiography of the escarpment which includes the rugged terrain. This forms a very steep morphology with a series of normal faults. The escarpment is mostly represented by basalts with intercalated ignimbrite-rhyolites.

The tectonic activities are associated with the formation of the Ethiopian Rift Valley, and the volcanic activity in the region was the main cause of the formation of valley, mountains and separate hills in the area surrounding the route corridor. Furthermore, the external activities are associated with a gorge system eventually dissecting the region into the formation of the valley through which major drainage patterns formed the Awash River flow.

3.5.2 Geology

This road project, Ankober-Dulecha, is located on the western escarpment of the Main Ethiopian Rift (MER). The rift escarpment is formed by a sequence of tertiary volcanic rock formation which is characterized by rugged relief with rock outcrops, deeply dissected creeks and channels but also by less steep areas with traced land. The project area largely marks the southern termination of the Ethiopian escarpment. It is a part of the Ankober border fault system, a major fault system that characterizes the structurally complex corner between the NE trending MER and the NS trending Red Sea Rift. (Wolfenden, et al., 2004)

In the area, some seismic activity has been also recorded by the EAGLE network of seismic stations in the period from October, 2001 to January, 2003 along with the complex Ankober border fault system. These events are responsible for numerous landslides and a formation of 12-15km long piedmont scarp in unconsolidated materials along the escarpment of the borkena graben with vertical differential displacement up to 2m, depth of 5-7m and width at the surface over 1m. (Keir, 2006)

The Ankober border fault system, which is related to all stages of rifting, is also defined by tertiary and quaternary mafic to felsic rocks (Figure 3.4). The volcanic rocks in this specific area include the following (Kazmin, 1979; Meshesha, e al., 1996):

Tertiary volcanic rocks: are mainly exposed on the plateau and escarpments of MER. The plateau volcanic rocks contents mafic lavas (Tarmaber Series) and felsic pyroclastic rocks (Debre Birhan–Gorgo ignimbrite). The rift escarpment series also mostly comprises volcanic rocks that were formed by the fissure eruption.

Quaternary volcanic rocks: dominate the eastern part of the project area, as rift–floor units and consist of two units; the Meteh bila basalt, the Dofan mafic volcanic rocks and the Fantale felsic rocks. The lavas are generally cut by step faults in further south and southeastern areas.

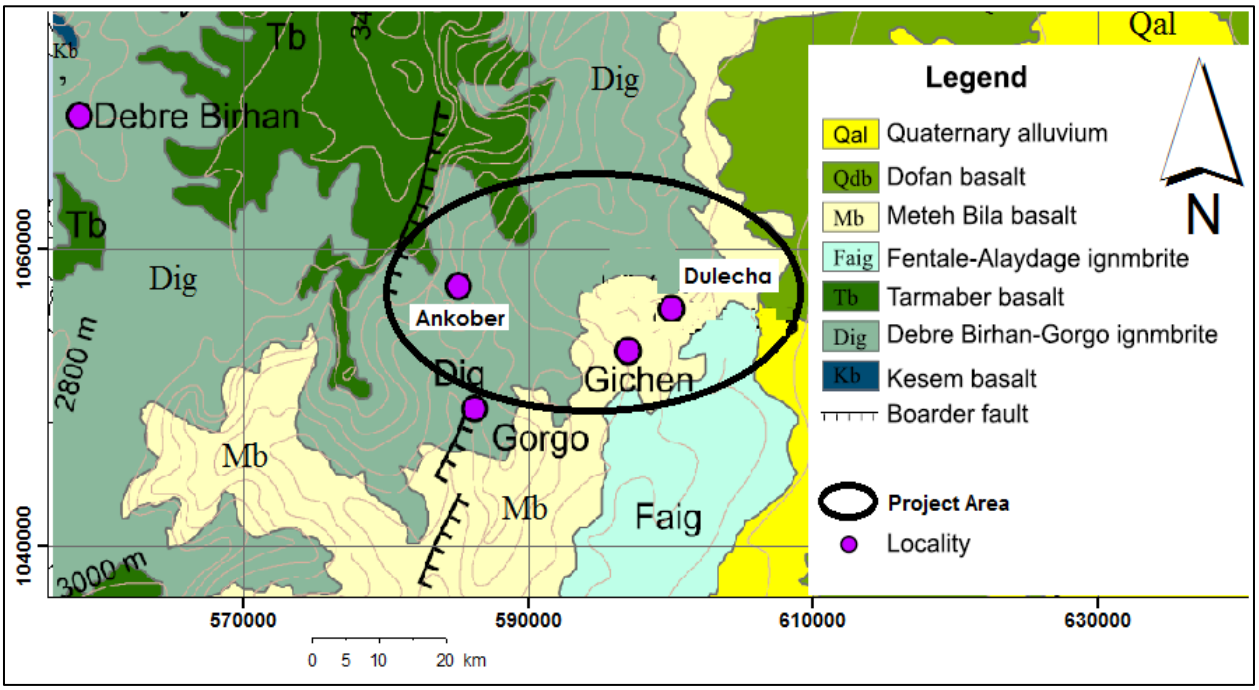


Figure 3. 4: Geological map of the project area (Shinjo, et al., 2017)

3.6 Climate and Soil

In general, climate is highly influenced by altitude and hence, rainfall increases as one goes from low to high elevation and vice versa for temperature. In this respect, this road project has a great altitude difference within the short distance and that causes the presence of significant climatic/weather differences over the road stretch.

The first 20km section of the road is located in a humid or highland zone with a mean annual rainfall of 1000–1200mm. The next section from km 20 to km 40 is situated in a semi–arid and

arid climatic zone which receives 600–700mm of annual rainfall. Rainfall usually occurs in the road project area in both Belg (March and April) and Kiremt (July, August and September) seasons. This is common mainly around the beginning of the project. The mean annual temperature of the road project varies from 20 to 37°C from the starting to the ending of the project (C–Tech Engineering Plc., 2018).

The dominant soil types for the first 37km of the road section are approximately reddish–grey silty gravel and clay with weathered ignimbrite and tuff. For the rest of the project area, the dominant soil types are brown clayey silt mixed with sand gravel and alluvial deposits.

3.7 Landslide History of the Area

The project area has a long history of landslides as the area is characterized by mountainous, escarpment and rugged terrain type with the presence of a significant amount of springs and stream water. Most unstable slopes of the area are associated with shallow groundwater and springs/seepages coupled with rugged terrain types. The springs are usually continuous marking impervious/saturated weak formation underlying often loose talus deposits and fractured competent volcanic formations. Moreover, due to the presence of high precipitation, especially around the beginning of the project, top loose talus deposits are often soaked and increase their weights while their shear strength becomes decreased and making them susceptible to progressive type failure.

The presence of a high population is also another factor for the existence of landslide in the area. This dense population has excessively cultivated the area and it is exposed to erosion for a long time. Even if the area has been deforested since a long time ago and the landslide has been encountered; currently, afforestation and natural resource protection work is being widely implemented.

Most of the noted landslides are located in tectonically formed sections with an extension character and predominantly in the direction of discontinuity (fault) orientations. This has mainly happened at those of tectonically formed depositions coupled with wet climate conditions and erosions facilitated by the presence of intense tectonism such as faults, shear zones and discontinuities.

4. INVESTIGATION AND ANALYSIS OF THE SELECTED FAILED SLOPES

4.1 General Description of the Slide Sections

A preliminary field visit was conducted to verify the failed slopes status in the Ankober–Dulecha asphalt road project. The affected slope sections' instability was visually assessed and the extent of the investigation required was discussed and recommended.

The alignment of the Ankober-Dulecha road upgrading project necessitated relatively deep cuts at places along the route to come up with the required design standard. Most of the sections also pass through topographically challenging terrain in which a significant amount of cuts with rock and rock mixed with soil layers are encountered. While excavation works were carried out, a considerable number of slope cuts were left critically unstable and tension cracks of 0.3 to 0.4m width and 0.5 to 1.8m depth were discovered at different sections of the road project in the upper part as well on the front side of the slope. Several tension cracks, vertical offset and scarps in the natural slope caused by the movement of residual blocks of soil mixed with some rock fragments were noticed.

Different types of materials are found along these cut slopes which include residual and colluviums soils in the upper part of the slope, dark to reddish-brown clay and fissured ignimbrite, light gray to dark gray, highly consolidated to welded tuffs bedded with columnar joint and fractured ignimbrite. Basalt is also dominantly exposed in the Ankober town parallel to the main road. Besides, there are many springs, and the groundwater is found in shallow depths which play a significant role in the generation of landslides in this road project.

Despite the loose soils on steep hillsides, landslides rarely occur around this area in the dry seasons and it is evident that the water pressures that build up in the ground, usually during the noticeable wet season, is expected to be the triggering factor for most of the sliding events of the area.

Even though several failed slopes were encountered at this road project, only the three critical and more of representative locations were selected for this investigation and analysis purpose.

a) Station 2+550, LHS

This section of the road is located in flat/rolling terrain area with shallow groundwater. Drilling was conducted by Civil Work Consulting Engineers Plc. (CWCE) located at station 2+550 and designed to determine the property of the subsurface materials, water conditions and to obtain samples for laboratory testing. At this location, a maximum depth of 10.0m was drilled from the original ground level. A prominent slip over about 40m length and extending back about 15m from the road has developed on the LHS of the cut slope. The failed mass comprises dark grey to black, highly fissured residual clay soils and greyish to white clay with gravel fragments. This could represent possibly debris of an old landslide.

The original road cut design comprises a slope ratio of 1.5:1 (H: V) without bench; however, the actual cut slope ratio is steeper than the original design around 1.3: 1(H: V). The height of the cutting varies from 4 to 7m high along the cut side. As shown in Figures 4.1, 4.2 and 4.3, a series of tension cracks have occurred which appear to have migrated back into the slope after each regressive slip.

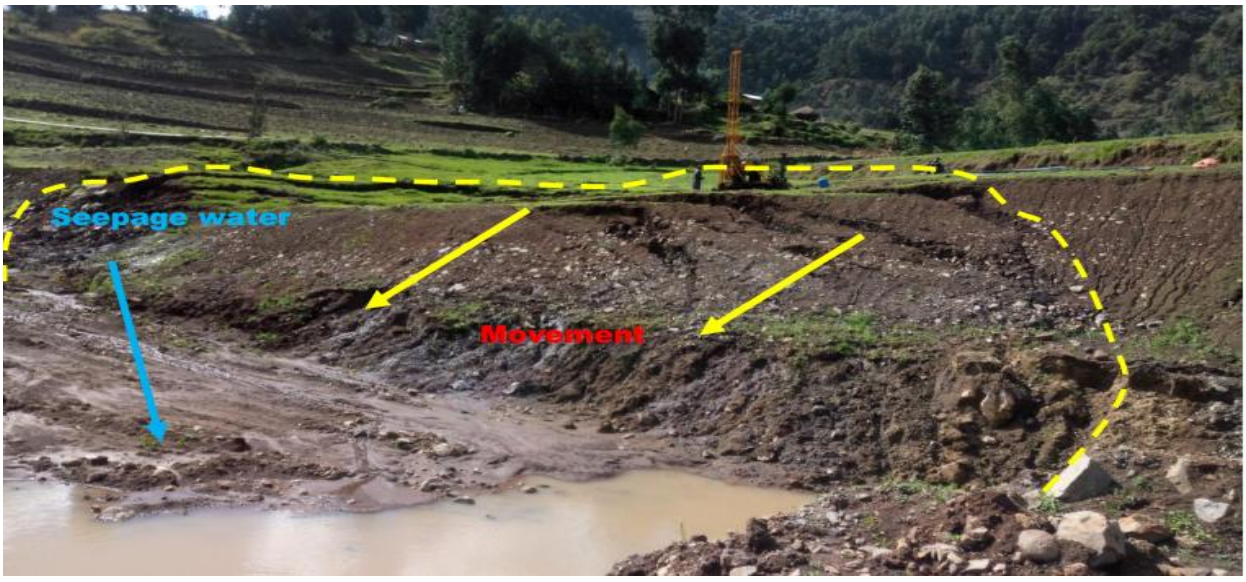


Figure 4. 1: Parallel tension cracks at the top of the slope which end up with a big landslide



Figure 4. 2: Loose, dark gray, clay soil mixed with basaltic fragments

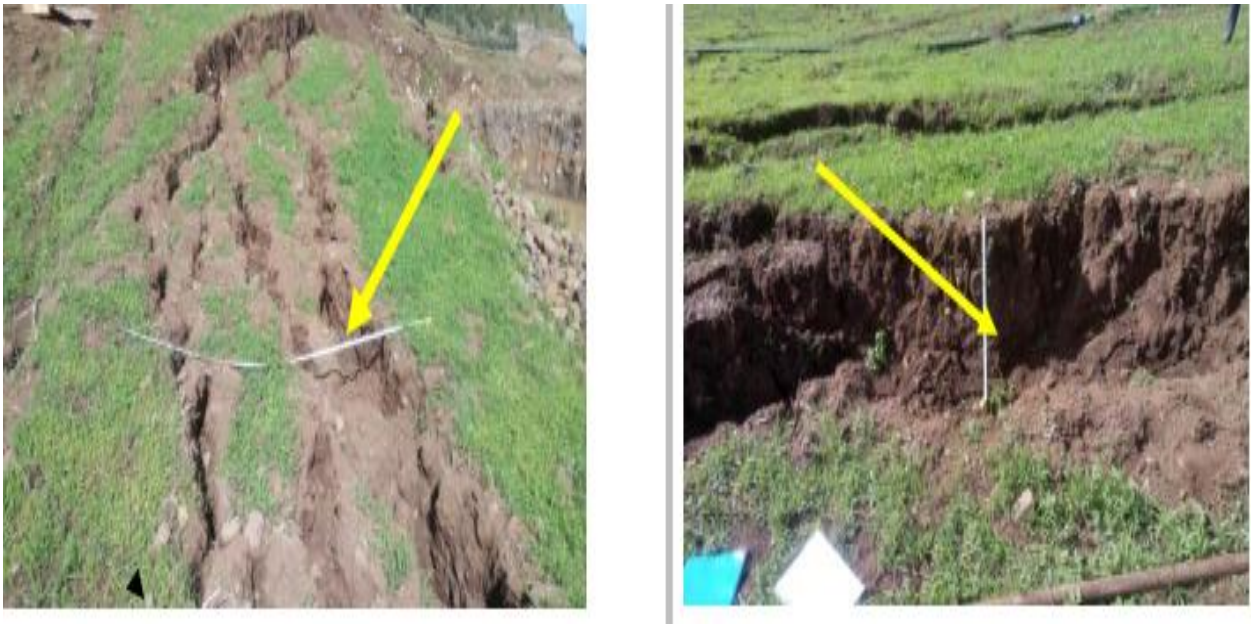


Figure 4. 3: Tension cracks and scarps noticed at the upper part of the slide mass

From visual observations (Figure 4.4), it is clearly evident that there is a significant amount of seepage evolved along the cut slope in the lower region of the excavated face.



Figure 4. 4: Water seepages at the intersection of the weak soil and decomposed rocks

b) Station 7+130, RHS

The designed side cut slope has two benches with a slope ratio of 1:1 and 1:1.5 (V: H) in the lower and upper parts of the slope respectively; however, the actual cut slope was not cut properly as per the original design and it failed before the finalization of the benchwork. The height of the cutting varies from 14 to 15m high around this station longitudinally. A prominent slip over 80m length and extending back about 17m (also, beyond this there are some tension cracks) from the road has developed on the RHS of the cut. In some portion of the failed slope, there are scarps which measure a height of 0.5 to 1.5m depth. In some sections, the slide materials bulge and flow down to the slope and reach the bottom of the road section. A significant amount of material is detached from the normal slope formation as illustrated in Figure 4.5.



Figure 4. 5: Detachment of the saturated mass and covering the excavated road level

As shown in Figure 4.6, tension cracks that are clearly visible on the top of the slope are observed and affected the nearby houses. Some portion of this landslide (from 80m) section is characterized by dark grey to light yellow and light brown to mottled pink colluvium soil with weathered ignimbrites.

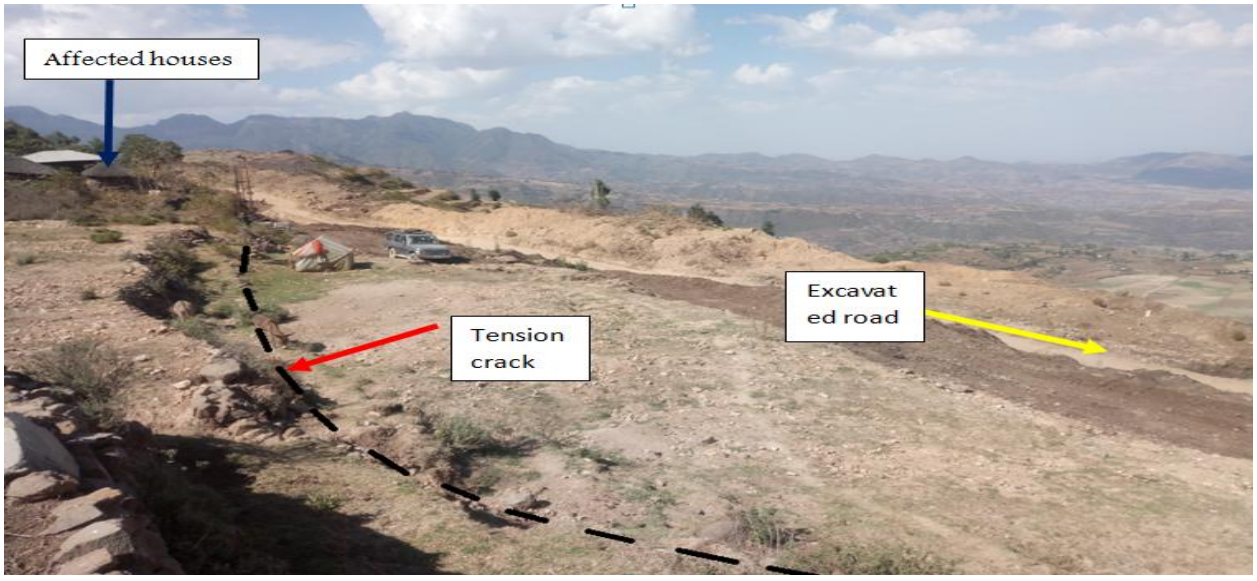


Figure 4. 6: Affected houses and tension cracks around station 7+130

The groundwater table is shallow as noted from the saturation of the soil at the ground level (probably from capillary water) and the presence of springs/seepages (Figure 4.7). The cut material is sensitive to infiltration/saturation when exposed to rainfall and surface streams (Figure 4.8).



Figure 4. 7: Spring/seepage at the side of the slope



Figure 4. 8: Flow of slope materials due to the presence of seepage coupled with surface streams

The geology of this slope comprises of slightly to highly decomposed ignimbrite and colluvial soils. As the depth increases down, the composition of weathered rocks and ignimbrites is increased. There is an interface between colluvial soil/earth material and the underlying weathered rock which was one of the major causes for failure (Figure 4.9). This is because as it is observed on the site, the upper weak soil (colluvial) is slide over the lower relatively competent ignimbrite formation.

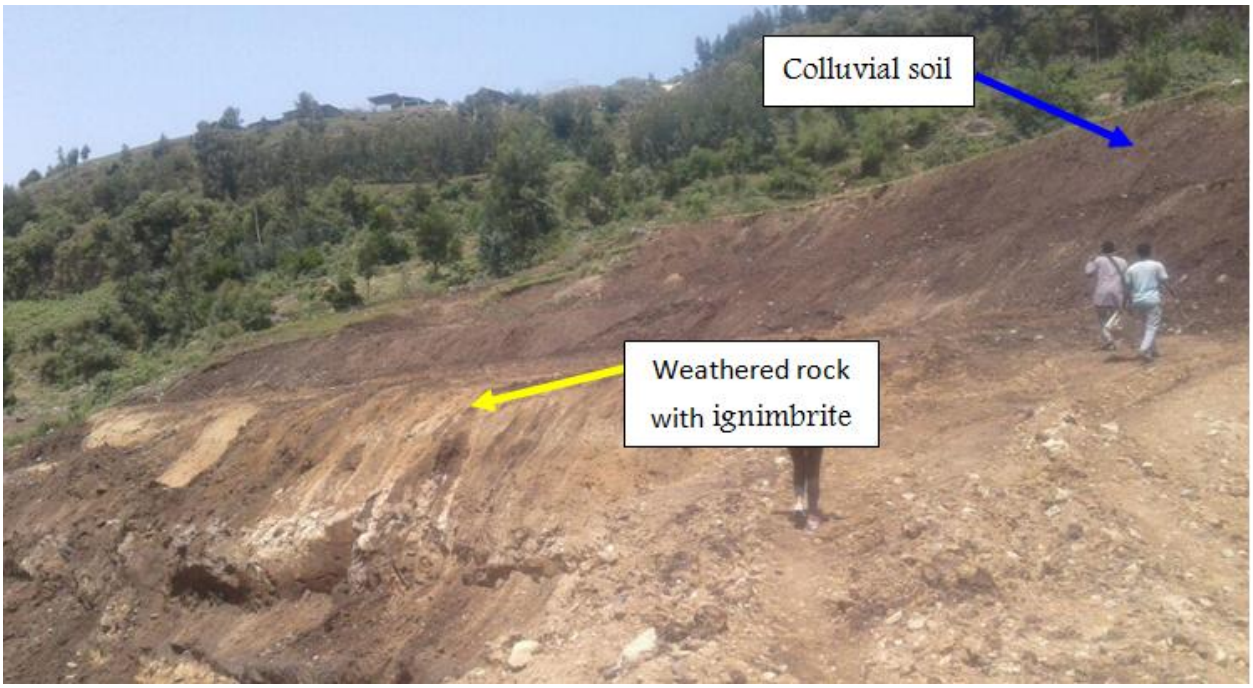


Figure 4. 9: The interface between colluvial soil and underlying weathered rock (after the removal of the slide mass)

c) Station 12+470, RHS

This section of the road is located predominantly on the rolling/flat terrain area. From a geologic perspective, the upper part of the slope is characterized mainly as the colluvium soil zone. The lower part of the slope is characterized as a highly weathered product of ignimbrite and gravelly lean and fat clay area. Drilling was conducted and is located at km 12+470 to verify the type of subsurface materials and water conditions. A maximum depth of 20 m was drilled at this location from the original ground level.

The cut section is located on a slope which is understood as an area less affected by previous landslides. The cut section is composed of an upper formation (highly plastic clay and some sandy lean clay) is weak and susceptible to slope failures. The other underneath formations, lean clay with sand and rock fragments and plastic clay associated with gravel/rock fragments, are relatively competent formations.

Based on the field observation, the cut section is affected by shallow successive failures. Active tension cracks are visible on the top and side of the slope cut (Figure 4.10 and 4.11). The cut slope has a slope height of 6.2m–7m with slope inclination around 37° with a slope cut ratio of 1:1.3 (V: H).



Figure 4. 10: Landslide movement at station 12+470, RHS



Figure 4. 11: Landslide movement at station 12+470 viewed at the top



Figure 4. 12: A clay soils mixed with weathered ignimbrite and gravels

4.2 Detailed Investigation of the Selected Failed Slopes

The detailed investigation process includes borehole drilling and preparation of borehole logs, field tests, sampling and laboratory testing, surveying data collection and groundwater level measurement.

4.2.1 Drilling

As it is recalled from the previous section, drilling of boreholes is carried out by CWCE. All the required drilling accessories were deployed. Each of the boring positions located using a total station.

On the logging sheet of each borehole, the Total Core Recovery (TCR), Solid Core Recovery (SCR), Rock Quality Designation (RQD), Average Fracture Spacing (AFS) and degree of weathering of the rock are indicated. All recovered materials from boreholes are kept in core boxes, properly labeled and photographed according to their depth of recovery. The log sheets and core box photographs are attached to Appendix D and E, respectively. (CWCE, 2018)

The soils encountered are classified according to the Unified Soil Classification System. Undisturbed and bulk samples of the subsurface materials were obtained at selected intervals to conduct laboratory tests. (CWCE, 2018)

4.2.2 In-Situ Testing

Standard Penetration Tests (SPT) are carried out along with the drilling activity. The equipment used to conduct the SPT is a standard device of 63.5kg-hammer weight with an automatic free drop height of 760mm. The total depth of penetration per test is 450mm, and the number of blows for the last 300mm penetration is recorded disregarding the penetration value for the first 150mm depth.

The driving of the split-spoon was terminated when N > 50 blows for any of the 15cm of penetration or when no advancement in penetration for 10 consecutive blows is recorded. N-values obtained are reported in the borehole logs without any correction. The N-value is corrected for the respective penetration influence factors. The corrected N-value is an essential parameter to determine the shear strength and other parameters in geotechnical engineering.

4.2.3 Groundwater Monitoring

As groundwater level affects the bearing capacity, triggered the landslides and determines the construction techniques, it was an integral part of the geotechnical investigation. Observations for groundwater were made during sampling, upon completion of the drilling operations at each boring location and it was monitored using a deep meter. In most failed slope sections, groundwater is found at very shallow depth. The groundwater location of the selected cut slope sections is shown in Table 4.1.

Table 4. 1: The location of the groundwater level along the borehole length

Station, km	Boring depth, m	Groundwater level below the ground surface, m
2+550	10	2.5
7+130 and 7+210	20	3.5
12+470 and 12+470	20	Not encountered

Areas of groundwater seepage are noted directly on the base of road cut sections. the primary source of this water is most likely from the infiltration of the surface and subsurface water coming from the hillsides of the slope.

4.2.4 Geotechnical Characteristics of the Failed Mass

a) Station 2+550

i. **A detailed description of the sampled soil**

Based on the data obtained from laboratory tests on representative soil samples and the core box along the slide section, the subsurface materials are approximately subdivided into two layers. The detail of the test results, classification using USCS, and description of the borehole of the sampled soils at this station are shown in Table 4.2 below.

Table 4. 2: Test result and description of the sampled soil at station 2+550

Layer	Test result (average values were taken for each layer with similar formation)	Classification of soil as per USCS	Borehole log description
Layer–1 (0–4m)	Natural Moisture Content, NMC: 24% Liquid Limit, LL:39% Plastic Limit, PL:21% Plasticity Index, PI:19% Pass no. 200 (0.075mm) sieve: 56% Pass no. 4(4.75mm) sieve: 59 %	The soil is dominantly gravely Lean Clay (CL)	The soil is characterized as a firm, light brown, Lean Clay (CL) with gravel

Layer-2 (4-10m)	Natural Moisture Content, NMC: (44% Liquid Limit, LL: 61% Plastic Limit, PL: 32% Plasticity Index, PI: 30% Pass no. 200 (0.075mm) sieve: 89% Pass no. 4(4.75mm) sieve: 99%	The soil is dominantly fat clay (CH)	The soil is mainly characterized as light grey, fat clay (CH) with some sand

ii. Plane profiles and different cross-sections of the failed mass

In this specific location of the failed slope, there is only one borehole (BH-A); however, to effectively describe failed mass, more number of drillings were required in the transverse and longitudinal directions. To minimize the problem of shortage of drilled boreholes, an ideal borehole is established (BH-B) in the RHS of the road by considering the outcrops from already excavated parts of the road. The plane profile of the affected area and boreholes location is shown in Figure 4.13.

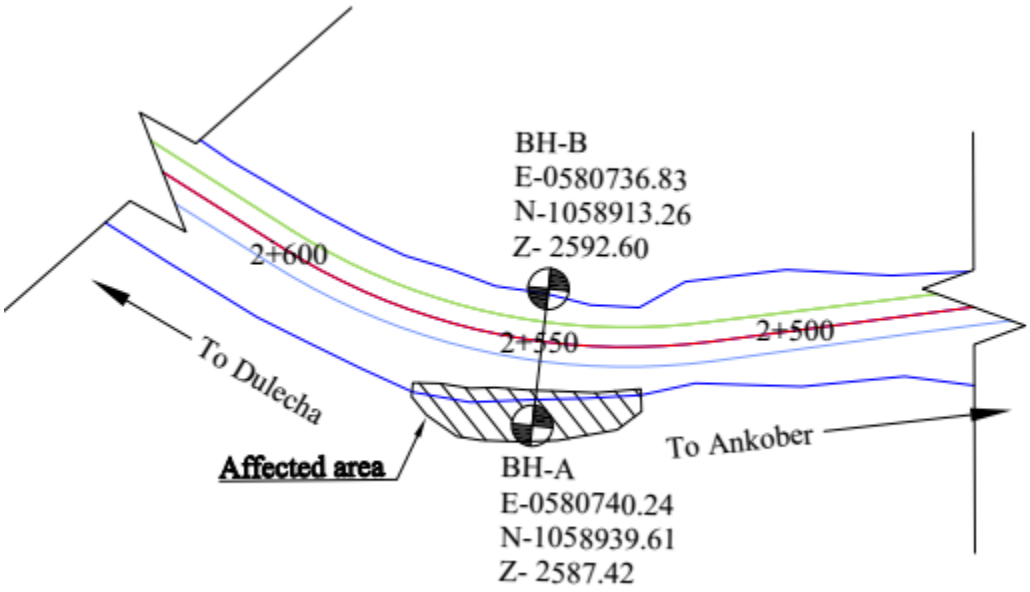


Figure 4. 13: Plane profile of the affected area and boreholes at station 2+550

As per the field and laboratory investigation result, the subsurface of this cut section is composed of two geotechnical layers. The transverse cross-section along borehole A and B is described in Figure 4.14.

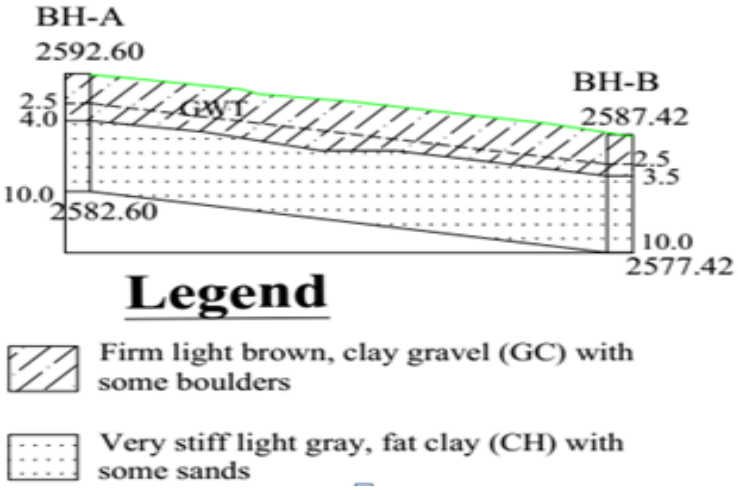


Figure 4. 14: Transverse cross-section at station 2+550

The cut slope of the road at this section, as shown in Figure 4.15 below, has the following geometrical or dimensional properties:

- The excavated slope height on the RHS is 6.59m and 2.92m on the LHS of the road.
- The LHS and RHS excavation limits (offsets) from the centerline of the road is 16.10m and 10.69m, respectively.
- Borehole A (BH-A) is located at 21.09m from the centerline of the road or 4.99m from the crown of the designed cut slope on LHS while BH-B is located at 10.69m from the centerline of the road on the RHS.

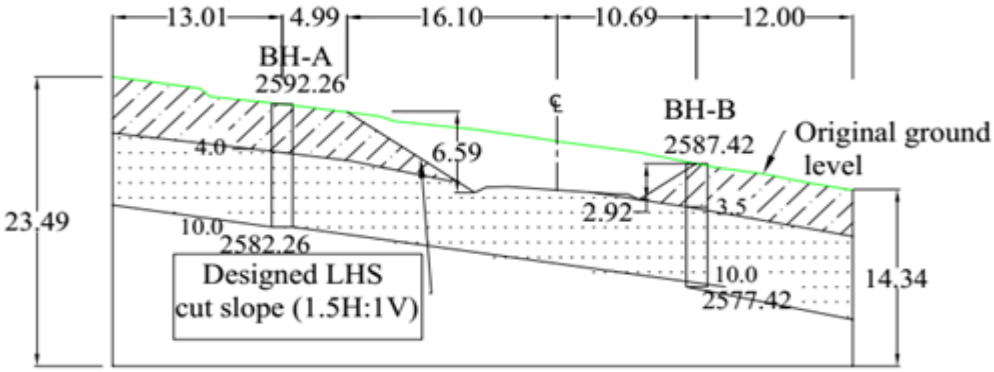


Figure 4. 15: Typical transverse cross-section and dimensions at station 2+550

The approximate slip surface is estimated (it is almost rotational type of slide) from the actual cut slope by considering the entry and exit of the slide mass. As shown in Figure 4.16 below, the entry of the slip surface is located at 8m from the crown of the excavated slope, and the exit is located at the toe of the cut slope.

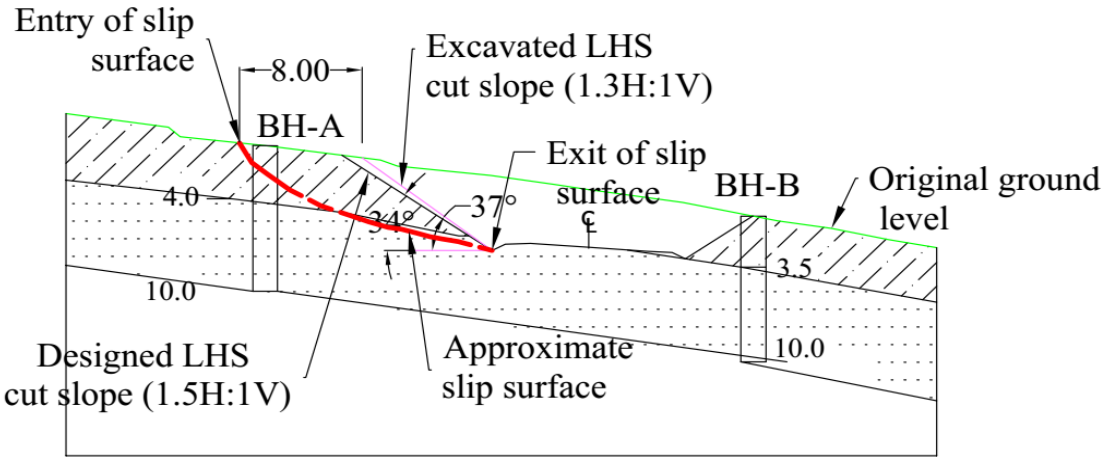


Figure 4. 16: Approximate slip surface at station 2+550

b) Station 7+130

i. A detailed description of the sampled soil

In general, the subsurface condition of this cut section is composed of four geotechnical layers as per the information gained from field and laboratory investigation. The test results, classification and borehole log description of the sampled soils at this station are shown in Table 4.3.

Table 4. 3: Test result and description of the sampled soil at station 7+130

Layer	Test result (average values for layers with similar formation)	Classification of soil as per USCS	Borehole log description
Layer-1 (0-6.5m)	Natural Moisture Content (NMC): 54% Liquid Limit, LL: 54% Plastic Limit, PL: 36% Plasticity Index, PI: 19% Passno.200(0.075mm)sieve: 73% Pass no. 4(4.75mm) sieve: 96%	The soil is dominantly elastic silt (MH) with sand	The soil is characterized as soft to firm, dark gray, elastic Silt (MH) with sandy and some angular to subangular gravels

Layer-2 (6.5-11m)	NMC: 36% Liquid Limit, LL: 41% Plastic Limit, PL: 23% Plasticity Index, PI: 26% Pass no. 200 (0.075mm) sieve: 59% Pass no. 4(4.75mm) sieve: 94%	The soil is mainly sandy lean clay (CL)	The soil is characterized as very stiff, brownish grey, sandy lean clay (CL) with a weathered product of Ignimbrite and some tuff.
Layer-3 (11-16m)	NMC: 29% Liquid Limit, LL: 45% Plastic Limit, PL: 26% Plasticity Index, PI: 19% Pass no. 200 (0.075mm) sieve: 87% Pass no. 4(4.75mm) sieve: 99%	The soil is mainly lean clay (CL)	The soil is mainly characterized as very stiff, pinkish to light brown, lean clay (CL) with a highly weathered product of Ignimbrite.
Layer 4 (16-20m)	NMC: 27% Liquid Limit, LL: 47% Plastic Limit, PL: 26% Plasticity Index, PI: 17% Pass no. 200 (0.075mm) sieve: 54% Pass no. 4(4.75mm) sieve: 99.8 %	The soil is mainly sandy lean clay (CL)	The soil is characterized as very stiff, light brown, sandy lean clay (CL) with a highly weathered product of Ignimbrite

To effectively describe the slope failure around station 7+130, the nearby borehole located at station 7+210 needs detailed characterization and hence, this cut section is divided into six lithological layers. The details of the test results, classification and borehole description of the sampled soils at station 7+210 are shown in Table 4.4 below.

Table 4. 4: Test result and description of the sampled soil at station 7+210

Layer	Test result (average values for layers with similar formation)	Classification of soil as per USCS	Borehole log description
Layer-1 (0-3.5m)	Natural Moisture Content (NMC): 42% Liquid Limit, LL: 53% Plastic Limit, PL: 36% Plasticity Index, PI: 17% Passno.200(0.075mm)sieve: 63%	The soil is dominantly elastic silt (MH) with sand	The soil is characterized as soft to firm, dark gray, sandy elastic Silt (MH) with some gravels

	Pass no. 4(4.75mm) sieve: 83%		
Layer-2 (3.5-6m)	NMC: 37% Liquid Limit, LL: 42% Plastic Limit, PL: 26% Plasticity Index, PI: 16% Pass no. 200 (0.075mm) sieve: 52% Pass no. 4(4.75mm) sieve: 99%	The soil is mainly silty sand (SM)	The soil is characterized as very stiff, light yellow, silty sand (SM) with a completely weathered product of tuff.
Layer-3 (6-9m)	NMC: 37% Liquid Limit, LL: 43 Plastic Limit, PL: 27 Plasticity Index, PI: 16 Pass no. 200 (0.075mm) sieve: 57% Pass no. 4(4.75mm) sieve: 99%	The soil is mainly sandy lean clay (CL)	The soil is mainly characterized as very stiff, pinkish-brown sandy lean clay (CL) with a highly weathered product of Ignimbrite.
Layer 4 (9-12m)	NMC: 27% Liquid Limit, LL: NP (none plastic) Plastic Limit, PL: NP Plasticity Index, PI: NP Pass no. 200 (0.075mm) sieve: 49% Pass no. 4(4.75mm) sieve: 92 %	The soil is mainly sandy silt (ML)	The soil is characterized as very stiff, pale white, sandy silt (ML) with a weathered product of Ignimbrite
Layer 5 (12-16m)	NMC: 33% Liquid Limit, LL: NP Plastic Limit, PL: NP Plasticity Index, PI: NP Pass no. 200 (0.075mm) sieve: 61% Pass no. 4(4.75mm) sieve: 99%	The soil is mainly sandy silt (ML)	The soil is characterized as very stiff, dark brown, sandy silt (ML) (CL) with a highly weathered product of Ignimbrite
Layer 6 (16-20m)	NMC: 34% Liquid Limit, LL: 45% Plastic Limit, PL: 27% Plasticity Index, PI: 18% Pass no. 200 (0.075mm) sieve: 81% Pass no. 4(4.75mm) sieve: 93 %	The soil is mainly sandy lean clay (CL)	The soil is characterized as very stiff, light brown, sandy lean clay (CL) with a highly weathered product of Ignimbrite

ii. Plane profiles and different cross-sections of the failed mass

In this section, there are two drilled boreholes longitudinally (BH–A and BH–B) on the affected part of the road. However, there is no borehole in the transverse direction which is critical to describe typical cross–section (perpendicular) of the affected road section. To minimize the problem of shortage of drilled boreholes in the transverse direction and increase the level of confidence when interpreting the geological and geometric information, an ideal borehole is established (BH–C) in the LHS of road. This is achieved by extending a line through BH–A and B up to crossing the opposite edge of the excavated road at BH–C. By considering geological information of BH–B and the outcrops from already excavated part of the road, BH–C is established. The plane profile of the affected area and boreholes location is shown in Figure 4.17. Furthermore, as shown in Figure 4.18, BH–C is projected to BH–C'. this is accomplished by considering geological information from BH–C, BH–A, and outcrops at this location.

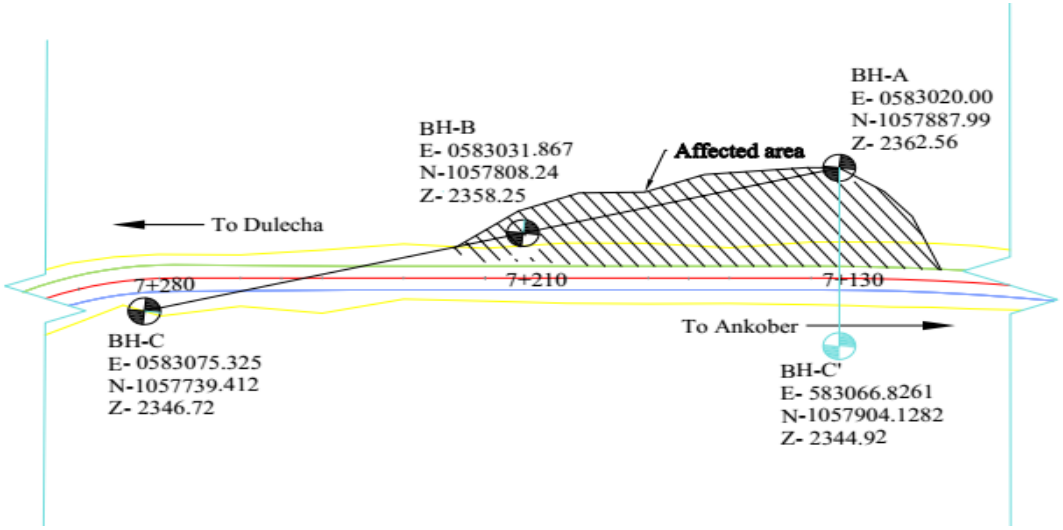


Figure 4. 17: Plane profile of the affected area and boreholes station 7+130

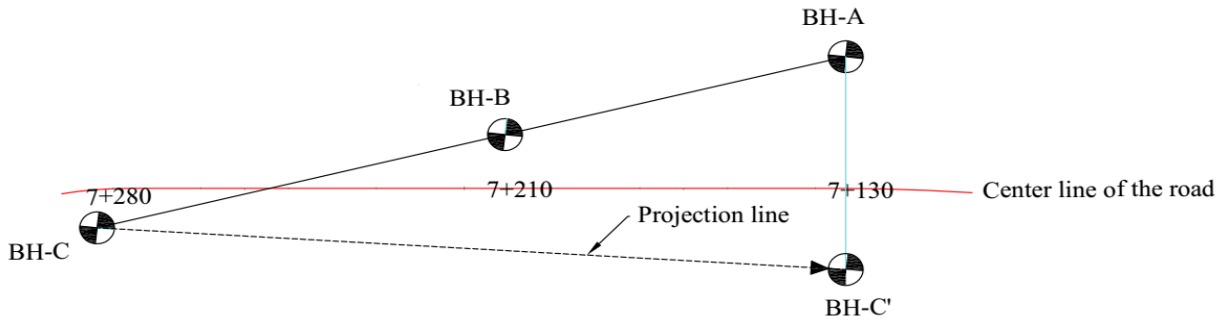


Figure 4. 18: Projection of BH–C to BH–C'

To interpret the subsurface geology of this cut stretch, cross-section drawings are taken along BH-A-B-C and A-C' and shown in Figure 4.19 and 4.20, respectively.

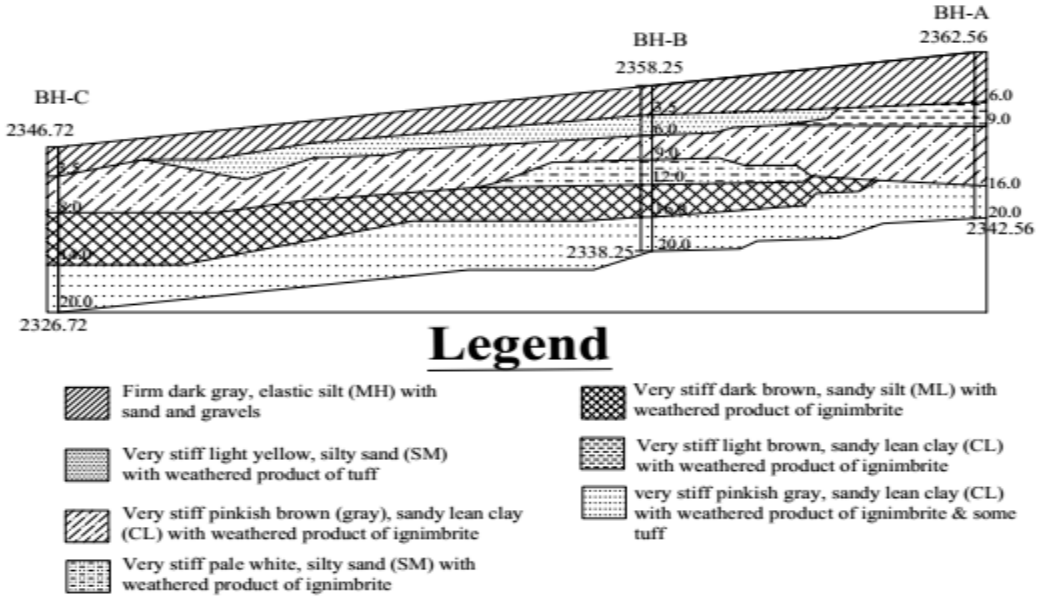


Figure 4. 19: Cross-section along BH-A-B-C (longitudinal cross-section)

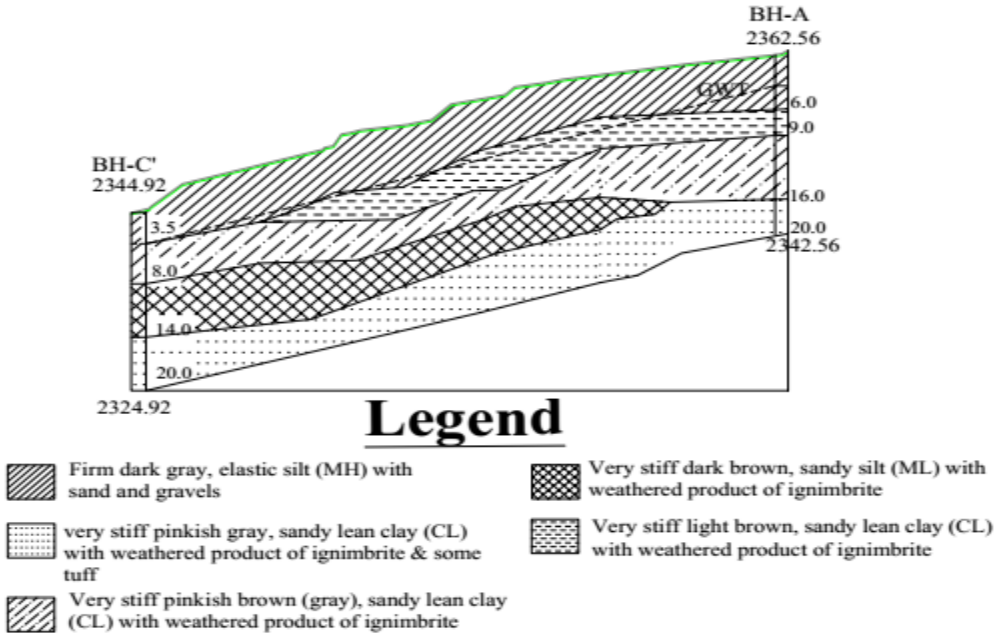


Figure 4. 20: Cross-section along BH-A- C' (transverse cross-section)

As shown in Figure 4.21 below, the cut slope of the road in this section has the following geometrical or dimensional properties:

- The excavated slope height on the RHS is 15.24m and 4.96m on the LHS
- The RHS and LHS excavation limits (offsets) from the centerline of the road is 23.87m and 10.36m, respectively.
- BH–A is located at 40.97m from the centerline of the road or 17.10m from the crown of the designed cut slope on RHS while BH–C' is located at 19.33m from the centerline of the road or 8.94m from the crown of the designed cut slope on LHS.

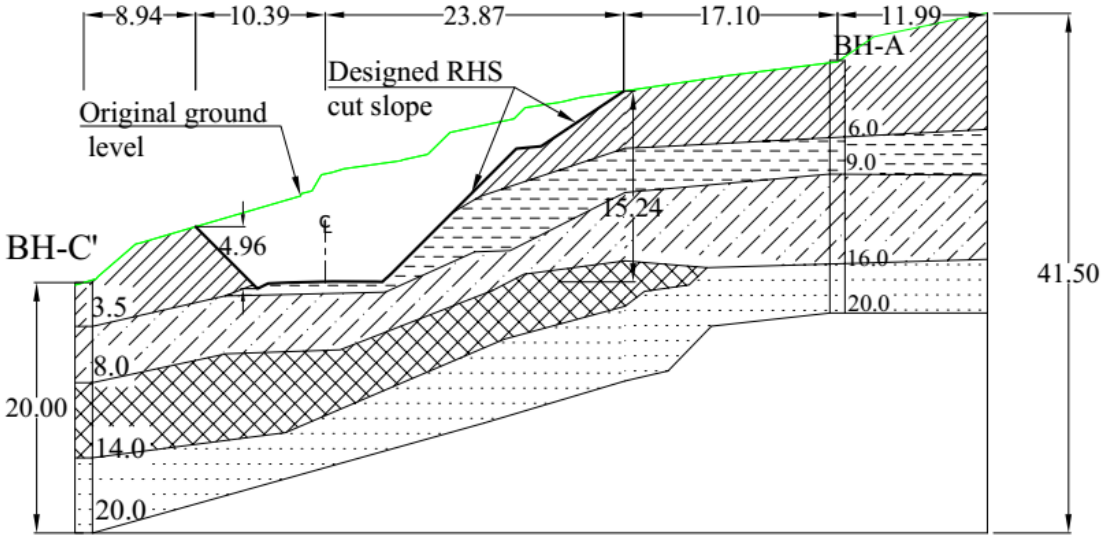


Figure 4. 21: Typical transverse cross–section with dimensions

The approximate slip surface of this cut section is delineated (it is almost translational type of slide with some flow) from the actual cut slope by considering the entry and exit of the slide mass. As shown in Figure 4.22 below, the entry of the slip surface is located at 18m from the crown of the excavated slope and the exit is located at 5.54m from the subgrade level of the excavated road when it is measured vertically.

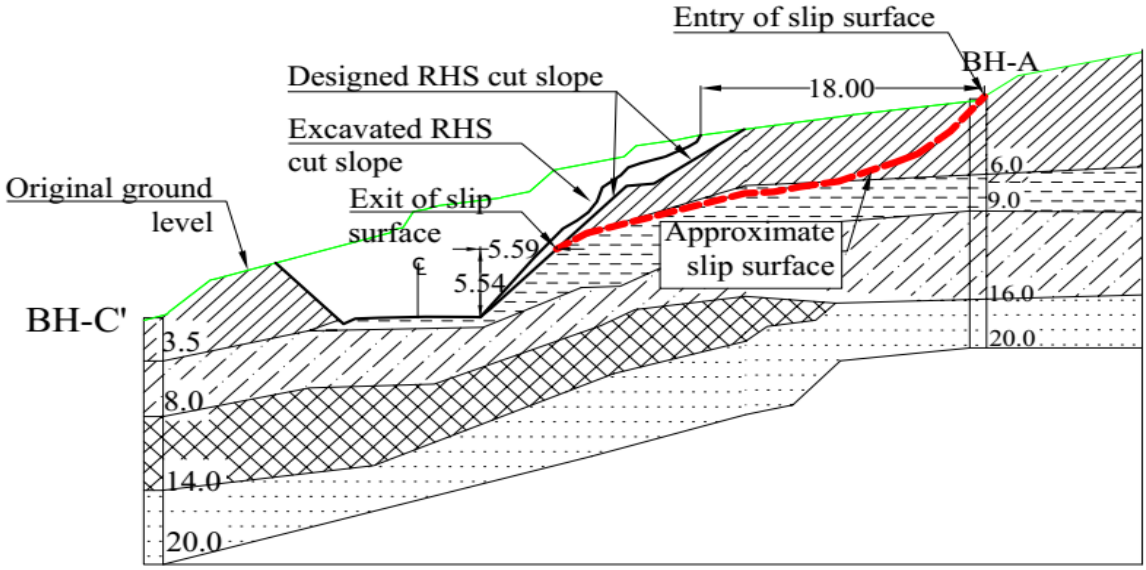


Figure 4. 22: Approximate slip surface

C) Station 12+470

i. A detailed description of the sampled soil

As per the information gained from field and laboratory investigation, the subsurface of this cut section is significantly composed of two lithological layers; however, to consider the effect of modulus of elasticity (its increasing nature) in the analysis part, it is divided into three layers. To efficiently describe this section, the nearby borehole located at station 12+490 needs detailed characterization and hence, this cut section is divided into four lithological layers for the sake of analysis. The details of the test results, classification and borehole description of the sampled soils at station 12+470 and 12+490 are shown in Table 4.5 and 4.6, respectively.

Table 4. 5: Test result and description of the sampled soil at station 12+470

Layer	Test result (average values were taken for each layer with similar formation)	Classification of soil as per USCS	Borehole log description
Layer-1 (0-8.5m)	Natural Moisture Content, NMC: 33% Liquid Limit, LL: 51% Plastic Limit, PL: 27%	The soil is dominantly lean clay (CL) with gravel	The uppermost 1m of the soil layer dominantly dark grey, top fat clay (CH).

	Plasticity Index, PI: 24% Pass no. 200 (0.075mm) sieve: 67% Pass no. 4(4.75mm) sieve: 76 %		Below 1m, the soil is characterized as a firm to very stiff, light brown lean clay (CL) with gravel and Ignimbrite boulders.
Layer-2 (8.5-20m)	Natural Moisture Content, NMC: 37% Liquid Limit, LL: 65% Plastic Limit, PL: 33% Plasticity Index, PI: 32% Pass no. 200 (0.075mm) sieve: 91% Pass no. 4(4.75mm) sieve: 97%	The soil is dominantly fat clay (CH)	The soil is mainly characterized as very stiff, dark grey, fat clay (CH) with highly weathered Ignimbrite and boulders

Table 4. 6: Test result and description of the sampled soil at station 12+490

Layer	Test result (average values for layers with similar formation)	Classification of soil as per USCS	Borehole log description
Layer-1 (0-3.5m)	Natural Moisture Content (NMC): 33% Liquid Limit, LL: 52% Plastic Limit, PL: 28% Plasticity Index, PI: 24% Passno.200(0.075mm)sieve: 84% Pass no. 4(4.75mm) sieve: 95%	The soil is dominantly fat clay (CH) with sand	The uppermost 1m of the soil layer dominantly dark grey, fat clay (CH). Below 1m, the soil is firm to stiff, light brown, lean clay (CL) with gravel and some boulders
Layer-2 (3.5-8m)	NMC: 24% Liquid Limit, LL: 45% Plastic Limit, PL: 24% Plasticity Index, PI: 21% Pass no. 200 (0.075mm) sieve: 66% Pass no. 4(4.75mm) sieve: 77%	The soil is dominantly gravely lean clay (CL)	The soil is characterized as a firm to very stiff, light brown to gray, brownish gyre, gravely lean clay (CL) with some gravels
Layer-3 (8-12.5m)	NMC: 35% Liquid Limit, LL: 64%	The soil is mainly fat clay	The soil is mainly characterized as very

	Plastic Limit, PL: 36% Plasticity Index, PI: 27% Pass no. 200 (0.075mm) sieve: 94% Pass no. 4(4.75mm) sieve: 97%	(CH)	stiff, light gray, fat clay (CH) with some gravels
Layer 4 (12.5–20m)	NMC: 15% Liquid Limit, LL: 43% Plastic Limit, PL: 22% Plasticity Index, PI: 21% Pass no. 200 (0.075mm) sieve: 72% Pass no. 4(4.75mm) sieve: 86 %	The soil is mainly sandy lean clay (CL)	The soil is characterized as very stiff, light brown to yellow, sandy lean clay (CL) with gravels and weathered product of Ignimbrite boulders

ii. Plane profiles and different cross-sections of the failed mass

Like that of the failed masses around station 7+130, in this stretch, there are two drilled boreholes in the longitudinal direction (BH–A and BH–B). However, there is no borehole in the transverse direction. Hence, an ideal borehole of BH–C is established bearing in mind the geological information of BH–B and the outcrops at this location. Then, as shown in Figure 4.24, BH–C' established by projecting BH–C and considering geological information from BH–C, BH–A, and outcrops of the excavated part of the road at this location. The plane profile of the affected area and boreholes location is shown in Figure 4.23.

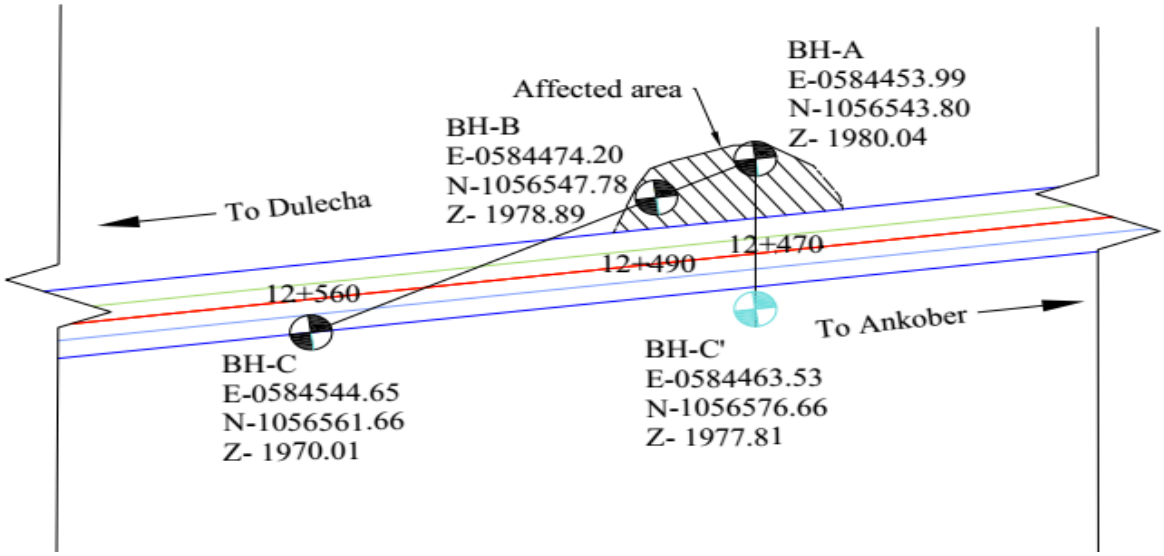


Figure 4. 23: Plane profile of the affected area and boreholes around station 12+470

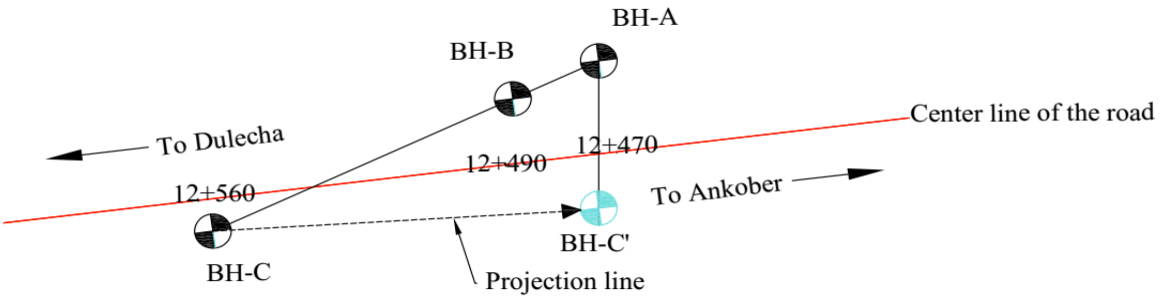


Figure 4. 24: Projection of BH-C to BH-C'

The cross-section drawings along BH-A-B-C and A-C' are shown in Figure 4.25 and 4.26, respectively.

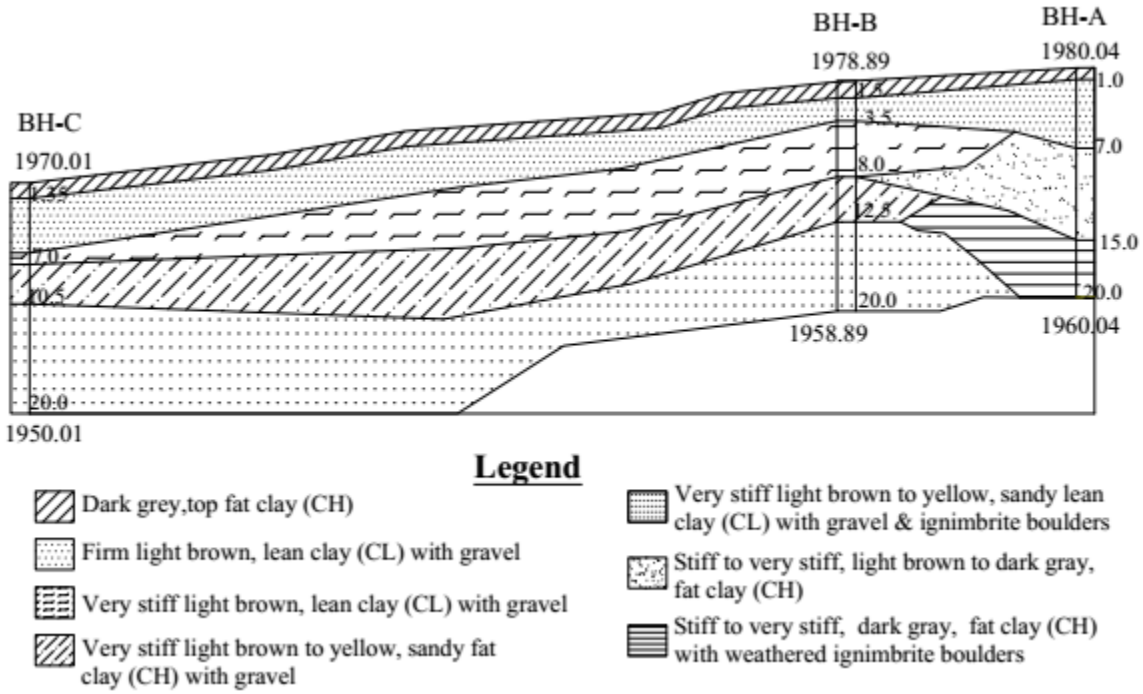


Figure 4. 25: Cross-section along BH-A-B-C (longitudinal cross-section)

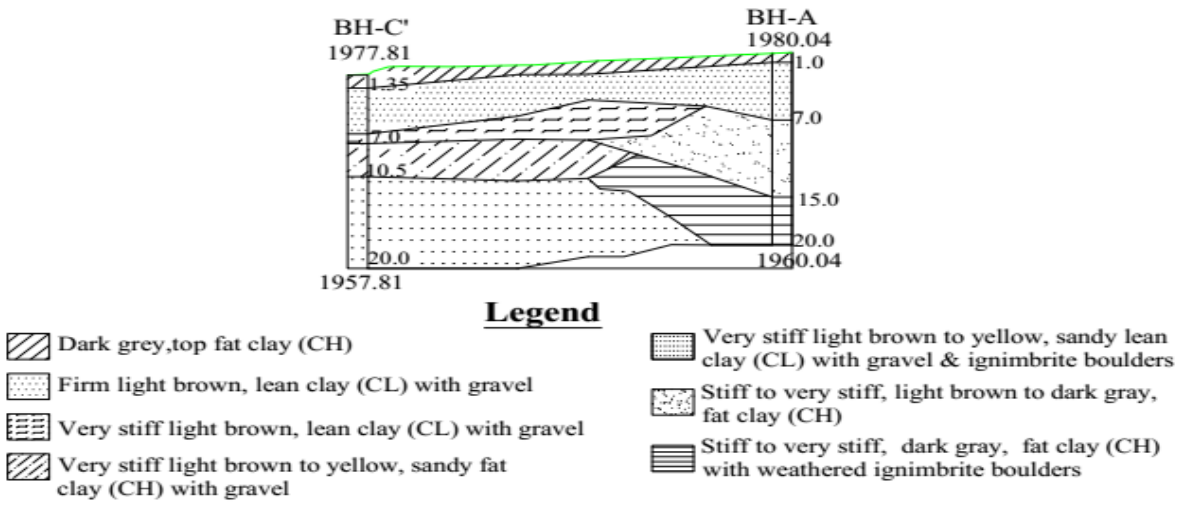


Figure 4. 26: Cross-section along BH-A-C' (transverse cross-section)

As shown in Figure 4.27 below, the cut slope of the road in this section has the following geometrical or dimensional properties:

- The excavated slope height on the RHS is 6.56m and 4.61m on the LHS
- The RHS and LHS excavation limits (offsets) from the centerline of the road are 14.62m and 12.92m, respectively.
- BH-A is located at 20.21m from the centerline of the road or 5.59m from the crown of the designed cut slope on RHS while BH-C' is located at 12.92m from the centerline of the road or at the crown of the designed cut slope on LHS.

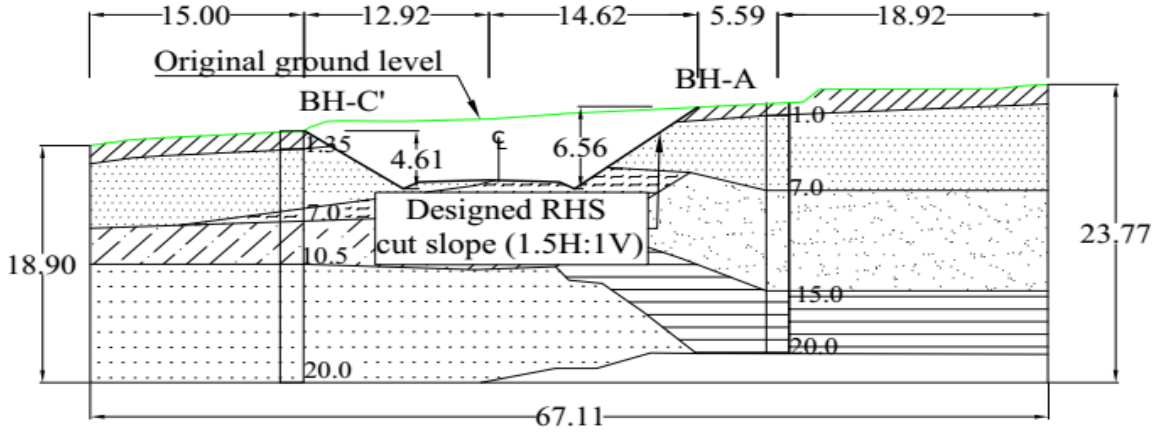


Figure 4. 27: Typical transverse cross-section with dimensions

The approximate slip surface is delineated (it is almost rotational type of slide) from the actual cut slope by considering the entry and exit of the slide mass. As shown in Figure 4.28 below, the entry of the slip surface is located at 7.36m from the crown of the excavated slope, and the exit is located at 0.95m from the subgrade level of the excavated road.

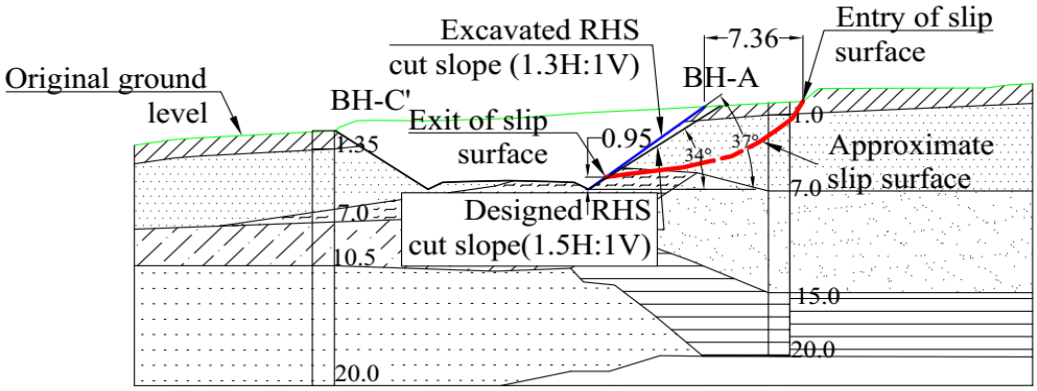


Figure 4. 28: Approximate slip surface

4.2.5 Sampling and Laboratory Testing

Disturbed and undisturbed soil samples from boreholes were selected and tested for classification, strength parameter determination and index properties identification of the slide mass. The laboratory test program includes atterberg limits, grain size analyses, moisture content, unit weight, specific gravity, shear strength parameters using direct shear test and other essential tests. The tests on representative soil samples were carried out by EJ Engineering plc. and the summarized results are presented in Appendix-A.

The shear strength parameters determined by EJ Engineering plc. were based on a direct shear test and this method of testing is mostly used for quickly drained soils (sand) and it is conducted through a predetermined shear plane without drainage control mechanism. Triaxial tests are employed for all soil types particularly for slowly drained soil types (clay) and are the most satisfactory one to measure the shear strength of soils for many engineering purposes even if it is the most expensive. Its principles are versatile and the procedures can be related to the numerous types of practical problems. It has its own remarkable advantages like controlling the magnitude of the principal stresses, measurement of pore water pressure and the controlling of drainage.

In this respect, it is evident that from the soil classification result that the soils are dominantly characterized as clayey soil; and to determine their shear strength parameters, triaxial testing is found to be important. Among the commonly used drainage conditions in triaxial shear tests (UU, CU and CD), the CU test is employed to find c' and Φ'. This test is the most common triaxial procedure as it allows strength parameters to be determined based on the effective stresses (c' and Φ') whilst permitting relatively a faster rate of shearing compared with the CD test. This is achieved by recording the excess pore pressure change within the specimen as shearing takes place.

Undisturbed soil samples were taken from each station at the place of accessible depth to withdraw with manpower. The tests were conducted at AAiT, Geotechnical Engineering laboratory center and the result is given in Appendix–B.

In the interpretation of the soil strength values, the modified failure envelop method or stress path representation system of the stress state using path parameters of q′ and p′ was employed to determine the effective shear strength parameters as it simplifies the difficulties of gating a tangent line for three points on three circles of Mohr's failure diagram (Arora, 1997).

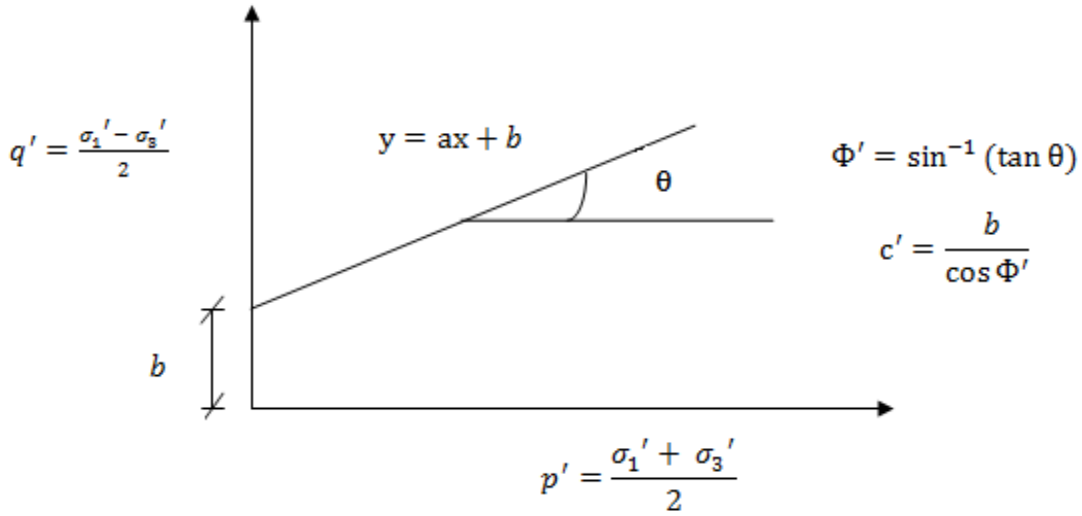


Figure 4. 29: Modified failure envelop

The Mohr–Coulomb failure criterion may be presented on the q′– p′ coordinate plane (Figure 4.29). This is based on the following relations obtained from the τ-σ (shear stress–normal stress) representation:

$$\frac{\sigma_1' - \sigma_3'}{2} = \left[\frac{\sigma_1' + \sigma_3'}{2}\right] \sin\Phi' + c' \cos\Phi' \rightarrow y = ax + b \quad (4.1)$$

$$a = \sin\Phi',\ b = c' \cos\Phi' \quad (4.2)$$

In the process of extracting of samples and testing this triaxial CU test, different problems are encountered, which include:

- The samples are extracted from the face of the failed slope which does not guarantee the collection of undisturbed sample
- The samples are extracted in the horizontal orientation which is less accurate than the vertical one
- At great depths, there were difficulties to extract soil samples with manually
- There was an interruption of power during the process of testing which affects the laboratory result
- The triaxial CU test equipment is not calibrated recently

As a result of this, the use of the SPT–N values was found necessary to determine the shear strength parameters of the soil.

4.2.6 Determination of Shear Strength Parameters from SPT N–values

The SPT is the most widely used in situ tests in geotechnical investigation and used to determine the in–situ properties of soil. It has been used to correlate different soil parameters like unit weight, relative density, angle of internal friction, compressive strength, and so on. It has also been used to estimate the bearing capacity of foundations and for determining the stress–strain modulus. Many correlations are devised between SPT N–value and ϕ as this SPT test is especially suited for cohesionless soils and only limited correlations are available between SPT N–value and undrained strength parameters of soils. However, there are no more correlations between SPT N–value with strength parameters of cohesive or c–ϕ soils. Due to this reason, the estimation of the strength parameters is carried out depending on the consistency ranges of the clay soil as recommended by Look (2007) as shown in Table 4.7 below.

Table 4. 7: Clay strength from SPT data (Look, 2007)

Material	Description	SPT-N	Strength (c_u) (kPa)
Clay	V. Soft	≤ 2	0-12
	Soft	2-5	12-25
	Firm	5-10	25-50
	Stiff	10-20	50-100
	V. Stiff	20-40	100-200
	Hard	>40	>200

Look (2007) has further recommended ranges values of the effective ϕ and c for the corresponding consistency of clay soils as stated in Table 4.8.

Table 4. 8: Effective strength of soil (Look, 2007)

Material	Description	c' (kPa)	ϕ' (°)
Clay	Soft-Organic	5-10	10-20
	Soft-Inorganic	10-20	15-25
	Stiff	20-50	20-30
	Hard	50-100	25-30

By considering the relations in Tables 4.7 and 4.8, shear strength parameters of the boreholes at hand are estimated from the corresponding SPT N-values. The parameters obtained from the laboratory tests are also used to be compared with the estimations from the SPT N-values. If the laboratory strength parameters fall within the minimum and maximum values that are determined from SPT N-value, it is considered acceptable; otherwise, the boundary (max. or min.) values that are nearest for the laboratory values are taken.

For each station, the uncorrected SPT N-values that are recorded at the time of drilling are presented in Appendix-D (Borehole diagram), and the corrected SPT N-values for the subject stations with their respective layers are presented in Appendix-C. As the SPT blow counts increase, the stiffness of the soil increases and its shear strength parameter values also increases. Layer designation and shear strength parameters for each station are described/presented below.

Station 2+550: the soil profile/layers and their corresponding shear strength parameter values are shown in Figure 4.30 and Table 4.9, respectively.

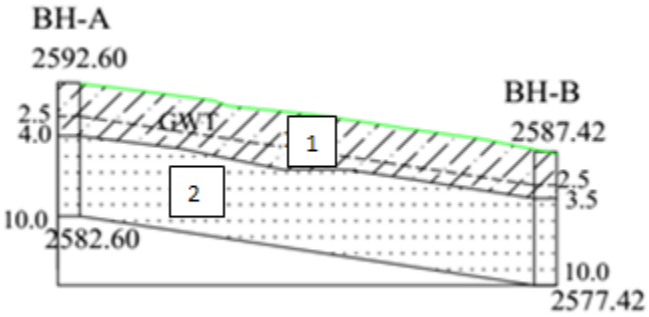


Figure 4. 30: Soil layers for the transverse cross-section along with station 2+550

Table 4. 9: Effective strength parameters for the transverse cross-section along with station 2+550

Layer	SPT-N	c'	$c'_{lab.}$	$c'_{ans.}$	ϕ'	$\phi'_{lab.}$	$\phi'_{ans.}$	Remark
1	Min=4	13	11.84	13	15	5.6	15	SPT-N from station 2+550 (BH-A)
	Max=10	25			20			
2	Min=18	33	35	35	23	5.7	23	
	Max=28	41			26			

Station 7+130: as it is described in section 4.2.4 (b) (ii), around this station of the failed mass, two boreholes are drilled (7+130 and 7+210) and the transverse cross-section at station 7+130 is established by considering the characteristics of these boreholes. Accordingly, the layers and the values of shear strength parameters are shown in Figure 4.31 and Table 4.10, respectively.

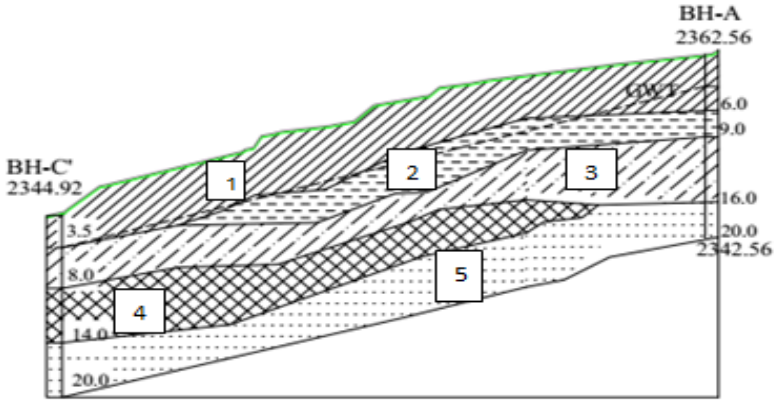


Figure 4. 31: Soil layers for the transverse cross-section along station 7+130

Table 4. 10: Effective strength parameters for the transverse cross-section along station 7+130

Layer	SPT–N	c'	$c'_{lab.}$	$c'_{ans.}$	ϕ'	$\phi'_{lab.}$	$\phi'_{ans.}$	Remark
	Min=5	15			16			
1	Max=12	27	16.1	16	21	6.9	16	SPT–N from station 7+130 (BH–A)
2	Min= max=55	50	33.4	50	30	10.5	30	
3	Min=max=58	50	–	50	30	–	30	
4	Min=max=58	50	–	50	30	–	30	SPT–N from station 7+210 (BH–B)
5	Min=max=58	50	–	50	30	–	30	SPT–N from BH–A

❋ For N> 40, the lower boundary of the range for c' and the upper boundary for ϕ' are taken

Station 12+470: based on the two boreholes drilled around this station of the failed mass (12+470 and 12+490), the layers and their corresponding shear strength parameter values are shown in Figure 4.32 and Table 4.11, respectively.

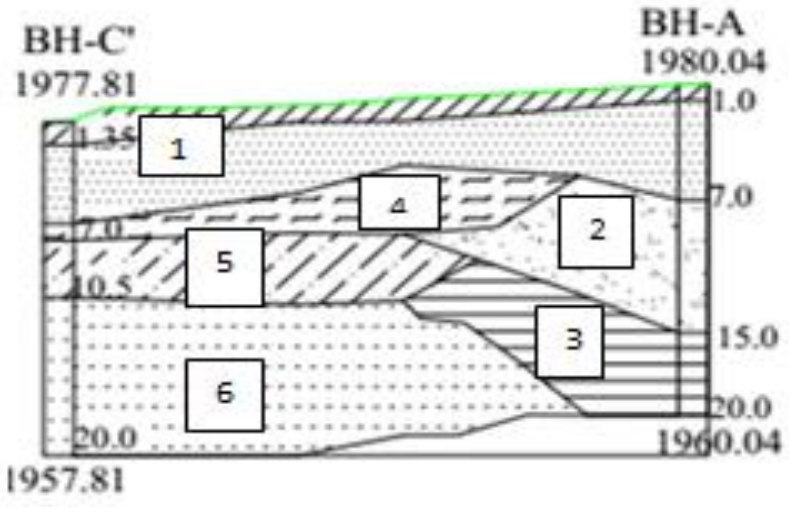


Figure 4. 32: Soil layers for the transverse cross-section along station 12+470

Table 4. 11: Effective strength parameters for the transverse cross-section along station 12+470

Layer	SPT-N	c'	$c'_{lab.}$	$c'_{ans.}$	ϕ'	$\phi'_{lab.}$	$\phi'_{ans.}$	Remark
1	Min=6	17	13.1	17	14	4.91	14	SPT-N from station 12+470 (BH-A)
	Max=12	27			17			
2	Min=25	38	33	38	22	8.3	22	
	Max=28	41			24			
3	Min=29	42	-	42	25	-	25	
	Max=35	46			27			
4	Min=max=42	50	-	50	30	-	30	SPT-N from station 12+490 (BH-B)
5	Min=max=60	50	-	50	30	-	30	
6	Min=max=60	50	-	50	30	-	30	

❋ For N> 40, the lower boundary of the range for c' and the upper boundary for ϕ' are taken

4.3 Analysis of the Failed Slope

The failure modes of almost any geotechnical structures are in 3D form and the analysis result using this 3D form reflects the real behavior of the situation. Even though the 3D analysis is applicable for any geotechnical structures, it is specifically implemented for conditions like short width geotechnical structures, soils subjected to concentrated loading, soil properties that vary significantly along the longitudinal direction of the slope and complex geometry. In contrary to 3D analysis, 2D analysis is more suited for the slopes having similar failure type and uniform soil formation. It also gives a conservative value of FS which is recommended for most slope design. Accordingly, for the specific problems encountered in this study, 2D analysis is employed.

4.3.1 Effect of Mesh Density

The model was fixed at the bottom of the horizontal boundary, total fixity, (u_x=0 and u_y=0) and for the vertical boundaries, left and right, only horizontal fixity was applied (u_x=0). PLAXIS 2D gives automatic boundary fixities using standard fixated.

There are different options of mesh densities in PLAXIS analysis, which may range from very coarse mesh to very fine mesh. Further, it is possible to refine the meshes around the significant load concentration areas or along the cut face of the slope (considering the high concentration of

load in the time of cutting). Accordingly, a sensitivity analysis was done, for the first station which is representative of the rest stations, for the different global coarseness densities concerning the factor of safety and number of elements (Table 4.12).

Table 4. 12: Effect of mesh density on the number of element and FS

Mesh type	Number of element	A global factor of safety
Very coarse	62	1.32
Coarse	99	1.22
Medium mesh	188	1.18
Fine mesh	444	1.11
Very fine	925	1.02
Refinement around the affected part of the slope	8188	1.01

From the result, as the mesh density increases, the number of elements increases and the factor of safety decreases. As the number of elements increases, the accuracy of the work increases even though it takes time and needs a high-performance computer. Generally, the choice of mesh density depends on the accuracy needed for the work and for this study; fine mesh with local refinement is used (Figure 4.33).

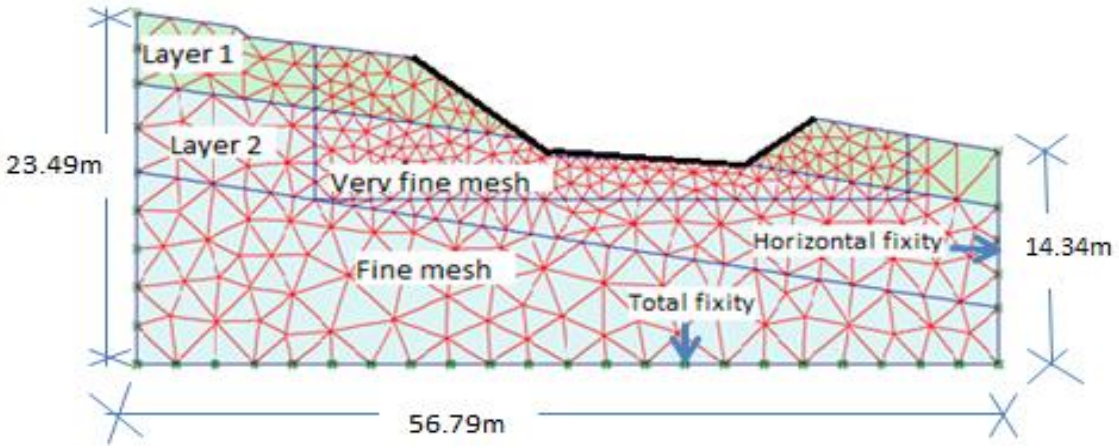


Figure 4. 33: Finite element mesh at station 2+550

4.3.2 Cases Considered for the Analysis

In slope stability analysis, it is essential to take consideration for different situations of the slope in accordance with the site condition. Accordingly the following were considered in this thesis:

Case-1: undrained analysis to account short term stability of the cut slope under actually investigated conditions of the subsurface geology and groundwater.

Case-2: drained analysis to consider the long term stability condition the cut slope. Slope stability analysis shall be conducted for both undrained and drained conditions as per the slope's loading condition; soil type and geotechnical structures (whether cut or embankment fill slope).

Case-3: slope stability analysis of the cut slope under high pore pressure (full saturation) to consider the possible rainfall saturation risk on the cut slopes. Accordingly, the full saturation of excess pore pressure condition is adopted to define the worst-case scenario during rainfall.

Case-4: slope stability analysis of the cut slope under earthquake loading to account for the seismic effect of the region as the study area is found at the margin of the Great Ethiopian Rift, which has a high potential of getting excited by an earthquake source.

4.3.3 Analysis Procedure of PLAXIS-2D

Input phase: Mohr-Columb model was applied to simulate soil behavior and material model. The K0-procedure was not used rather gravity loading was used to account for the inclination of the ground.

Calculation phase: after mesh generation, the calculation phase started through building up of initial stresses within the deposit soils. For phase 0, the total multiplier was used and for the other cases /steps, plastic calculation type was used except the last step which is phi-c reduction.

Output phase: in the last phase, phase 3, the calculation type is applied as a safety factor to determine collapse loads and calculate the stability. The program uses ΣMsf to specify the increment of strength reduction.

4.3.4 Output of the Analysis

The stability of the cut is analyzed on the notion of Factor of Safety (FS) against sliding to indicate the potential slide. Most parameters are determined from laboratory and site investigation results except the Poison's ratio, which was taken from presumptive values based on the soil types.

a) Station 2+550

This section is mainly composed of gravely lean clay soils (0–4m) and with the next underneath thick fat clay with some sand deposit (4–10m). The cut slope has a height of 4m–7m with a slope inclination about 37° (cut slope ratio of 1:1.3, V: H) which is steeper even than the original design cut slope inclination of 34° (about slope cut ratio of 1.5:1, H: V). There are springs and surface streams along the slope, and the groundwater table is found at 2.5m below from the ground surface. The designation of the soil layers with a number is shown in Figure 4.34 to simply understand the layers with their properties in the course of analysis.

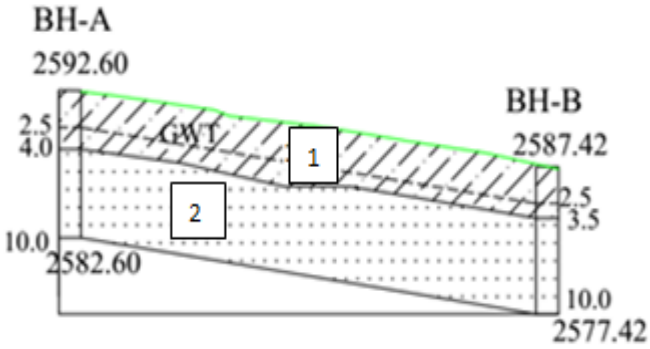


Figure 4. 34: Layer designation for the cross–section at station 2+550

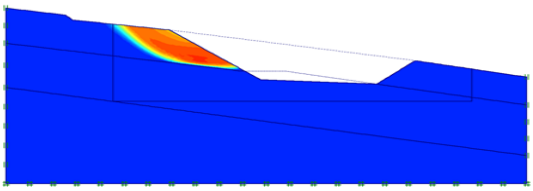
Table 4. 13: Input parameters for PLAXIS software at station 2+550

Layers No.	Description of layers	γ_b [kN/m^3]	c' [kN/m^2]	ϕ' [°]	E' [kPa]	ν' [–]	GWT [m]
1	Lean clay with gravel	17.15	13	15	5,750	0.35	2.5
2	Fat clay with sand	17.4	35	23	16,211	0.35	

- Note that the value of modulus of elasticity was evaluated from the SPT–N value and the procedure is attached in Appendix–C.

For the three stations, the analyses were done for the different cases that are mentioned in section 4.3.2. Case-1 for the normally investigated slope with the undrained condition to consider the short-term condition. Case-2 for drained condition as the investigated cut slope soils are mainly a clay soil then considering the long-term effect is necessary. Case-3 for full saturation condition; since the study area is frequently exposed to rainfall, and subsurface springs (stations 1 and 2), the chance of excessive saturation is highly expected. Case-4 for seismic effect; even if the area is found in the margin of the MER, there was no evidence/sense of seismicity during the failure of the cut slopes and not accounted for in this analysis, but it is considered in the remediation analyses to account for the probability of occurrence. The analysis results of these conditions are shown in Table 4. 14 for station 2+550.

Table 4. 14: Analysis result for different conditions at station 2+550

Conditions	Undrained	Drained	Full saturation
The sample failure mode (full saturation condition)			
FS	1.21	1.024	0.984

The actual cut slope analysis showed that the slope is not safely stable for undrained condition (FS=1.21), on the verge of failure in drained condition (FS=1.024) and it is further unstable under full saturation condition (FS=0.984). It is found that the drained condition is more critical than the undrained condition (FS_d =0.984< FS_u = 1.024). Among the three cases, the solution should be made based on the full saturation condition.

b) Station 7+130:

This cut section is located in the middle of the previous slide mass that shows some lobate and triangular facets. This cut section is composed of the upper highly plastic fine soil which has 6.0m thickness. The other underneath formations, lower lean clay with sand/rock fragments and sandy

silt which is relatively competent. The slope is steep and there are clear detachments and new cracks formed with the new slope cutting activities. This cut slope has a slope height of 14.7–15.2m with a slope inclination of 45° for lower slope and the bench is not properly provided. There are springs along the slope and the groundwater table is approximately situated at 3.5m from the ground surface. The designation of the soil layers with a number is shown in Figure 4.35.

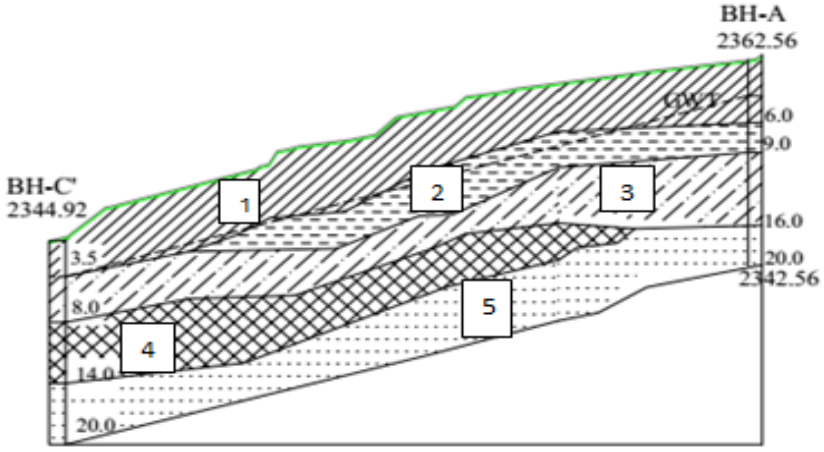
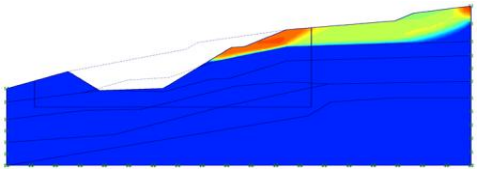


Figure 4. 35: Layer designation for the cross-section at station 7+130

Table 4. 15: Input parameters for PLAXIS software

Layer No.	Description of layers	γ_b [kN/m^3]	c' [kN/m^2]	ϕ' [°]	E' [kPa]	ν' [–]	GWT [m]
1	Elastic silt with sand	17.10	16	16	7,893	0.35	3.5
2	Sandy lean clay	17.12	50	30	44,564	0.35	
3	Lean clay	17.58	50	30	55,465	0.35	
4	Sandy silt	18.16	50	30	61,773	0.35	
5	Sandy lean clay	17.63	50	30	65,478	0.35	

Table 4. 16: Analysis result for different conditions at station 7+130

Conditions	Undrained	Drained	Full saturation
The sample failure mode (full saturation condition)			
FS	1.29	1.083	0.991

It can be observed that the cut slope is not satisfactorily stable in undrained condition (FS_u =1.29), on the verge of failure in drained condition (FS =1.083) and it is further unstable under full saturation condition (FS=0.991). It is found that the drained condition is more critical for this cut slope than the undrained condition (FS_u =1.29> FS_d =1.083). The remedial solutions should be proposed based on a full saturation condition since it is critical among the three cases.

c) Station 12+470

This cut section is located in the slope which is interpreted as an area less affected by previous landslides. It is composed of the upper formation of highly plastic clay. This formation is weak and susceptible to slope failures. The other underneath formations of plastic silt associated with gravel/rock fragments which are relatively competent formation. The cut slope has a height of 6.2–7m with a slope inclination around 37° which is steeper even from the original design cut slope inclination of 34°. The groundwater was not observed within the drilled depth (0–20m). The designation of the soil layers with a number is shown in Figure 4.36 below.

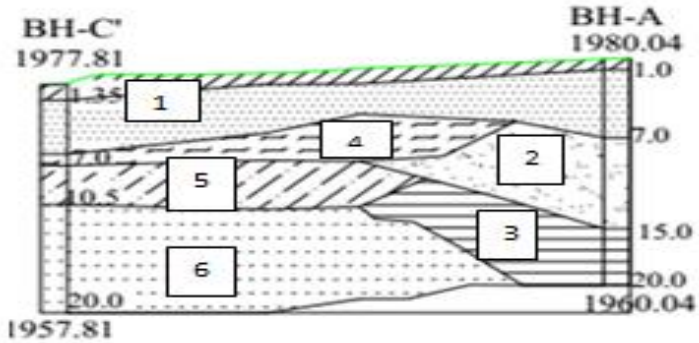
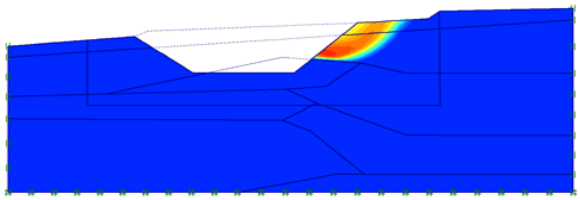


Figure 4. 36: Layer designation for the cross-section at station 12+470

Table 4. 17: Input parameters for PLAXIS software

Layer No.	Description of layers	γ_b [kN/m^3]	c' [kN/m^2]	ϕ' [°]	E' [kPa]	ν' [-]	GWT [m]
1	Graverly lean clay	17.17	17	14	10,912	0.35	Nil
2	Fat clay with sand	17.47	38	22	34,864	0.35	
3	Fat clay with sand	16.96	42	25	53,105	0.35	
4	Graverly lean clay	16.95	50	30	33,966	0.35	
5	Fat clay with gravel	17	50	30	75,422	0.35	
6	Sandy lean clay	17.1	50	30	106,032	0.35	

Table 4. 18: Analysis result for different conditions at station 12+470

Conditions	Undrained	Drained	Full saturation
A sample failure mode (full saturation condition)			
FS	1.341	1.161	1.159

It can be noted that the slope in the undrained condition is in stable condition even though it is not satisfactory. The slope in drained and full saturation conditions is on the verge of failure. Drained condition is critical than undrained condition. In the proposition of remedial measures for this station, saturation condition should be considered to account for the worst-case scenario as it is critical among the three cases.

Generally, as it is shown in the three stations analyses result, the long term condition (drained condition) is more critical than short term (undrained condition) and it assures the decrement of FS through time after the excavation of the slope due to pore pressure equilibration. That is why most cut slopes are failed after a time.

4.3.5 Comparison of the Actual and Analyzed Slip Surfaces

It is endeavored to measure the actual slip surfaces of the failed slopes on the ground based on the entry and exit of the slide masses. When they are compared with the analyzed slip surface, there are some deviations.

For station 2+550: in this station, the slip surface actually on the ground and obtained after analysis are almost coincided with each other except at the toe of the excavated road, the actual slip surface is overestimated and is shown in Figure 4.37 below.

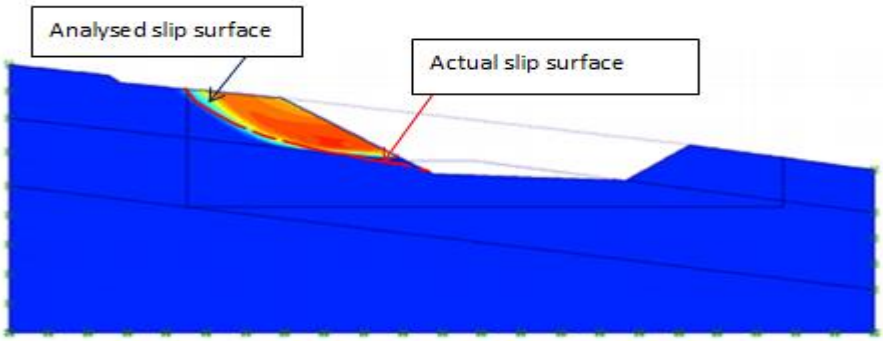


Figure 4. 37: Actual and Analyzed slip surfaces at station 2+550

For station 7+130: here, even though the two slip surfaces have a similar nature, the analyzed slip surface extends back than the actual one. This is due to, on the actual ground, there are some tension cracks far from the main affected part of the slope and the actual slip surface measurement was not accounted for these tension cracks. The analyzed slip surface follows the upper weak soil layer and both slip surfaces are shown in Figure 4.38.

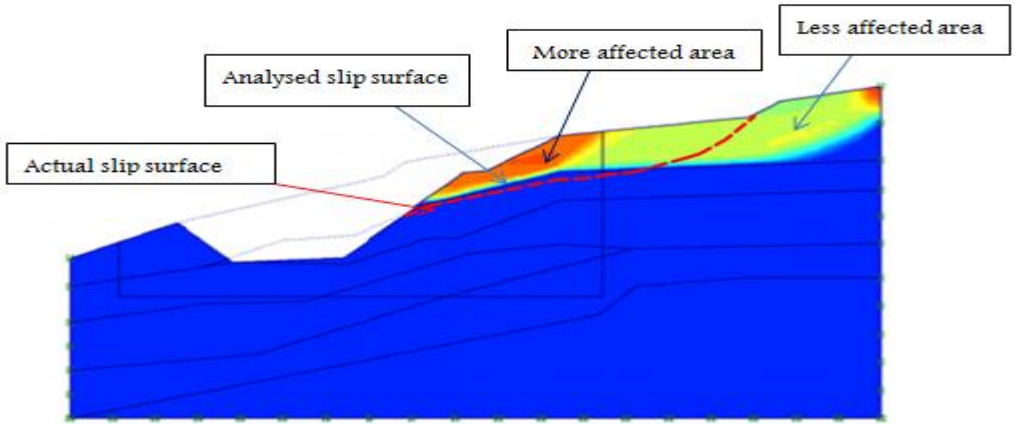


Figure 4. 38: Actual and analyzed slip surfaces at station 7+130

For station 12+470: in this station, the slope is not significantly affected by landslide and there are tension cracks on the top and a few on the said of the excavated slope. On the top of the excavated slope, the severity of the crack decrease as one goes from the crown of the slope to the back direction and the actual slip surface was measured at the very thin crack which is far from the crown. Due to this scenario, the actual slip surface is overestimated and deviated from the analyzed slip surface. Both actual and analyzed slip surfaces are shown in Figure 4.39 below.

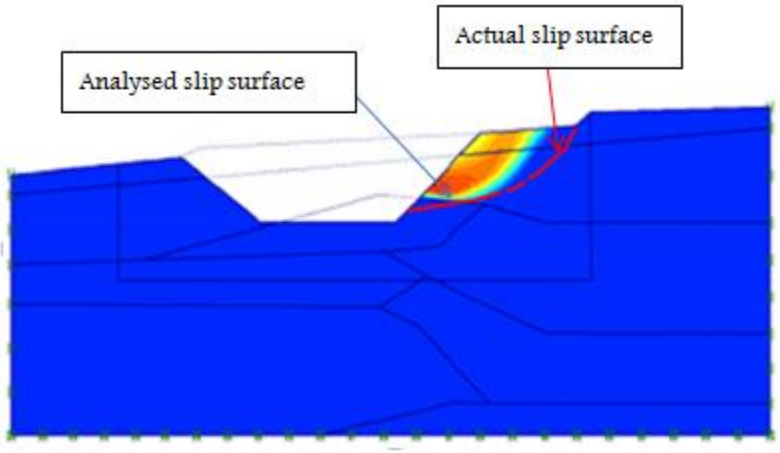


Figure 4. 39: Actual and analyzed slip surfaces at station 12+470

5. REMEDIAL SOLUTIONS

There are always two choices to improve the stability of any slopes, which are increasing resistance or decreasing driving forces or moments. Stabilization of the cut sections might be generally difficult because of its complex geological and geomorphological processes and patterns of the actual site conditions. On some occasions, one remedial measure may be sufficient to minimize the effect of a cut triggered landslide. However, most of the time, it is recommended to use a combination of methods to effectively manage the instability problems.

In the selection of appropriate remedial measures, consideration should be made to account for factors like engineering feasibility, cost–effectiveness and environmental acceptability of the methods. In the prevention/remediation of the landslide, the countermeasures may be either Control or Restraint work. (Abramson, et al., 1996)

i. Control works: are methods employed to avoid or reduce the triggering factors like provision of furrow ditch at the crest of the slope, flattening or removal of the slide mass or slope, artificial change on natural /cut slope angle and height.
ii. Restraint works: are methods employed to stabilize the landslide by the construction of structures like provision of different retaining walls to counteract the driving force.

Depending on the investigation and analysis of the slides sections, in this thesis study, some specific remedial measures are proposed for respective slide locations.

5.1 Station 2+550, LHS

Reasons for failure:

From the result of investigations and analyses, the main reasons for the failure of this cut slope were:

- Even though the original cut slope was designed with a grade of 1.5:1 (H: V), the actual cut slope is constructed with an average cut slope ratio of 1.3:1 (H: V). This is relatively steep for the prevailing materials and it has reduced the stability of the slope.
- The presence of springs and surface streams along the slope and the shallow groundwater table affects the stability. Further, the occurrence of an extraordinary rainfall after the completion of cut slope construction had seriously aggravated the slope failure.

- The slope formation is composed of gravely clay and thick clay deposit and hence, these units of the soils are weak and susceptible to failures mainly upon wetting.

Remedial Measures:

Considering the overall investigation and analysis result discussed before, some specific remedial solutions are suggested for this cut section.

Effect of slope flattening: in the previous section, the stability analyses were done based on the actual conditions that were investigated on the site and it is evident that the actual cut slope is relatively steeper than the design cut slope. In this respect, checking the results of stability with the original design slope and further flatter slopes are required to understand the effect of slope inclination. Hence, the original design cut slope ratio of 1.5:1 (H: V) and 2:1 (H: V) were taken to understand the same and the result is shown in Table 5.1. in the analysis of this remedial measure type:

- Full saturation condition is considered (to account for the worst-case scenario)
- Seismicity: the study area is located in the seismic hazard zonation of zone four and its Peak Ground Acceleration (PGA) value is 0.15g. This value is used for all stations to consider the earthquake effect of the area. (Ministry of Construction, 2015)

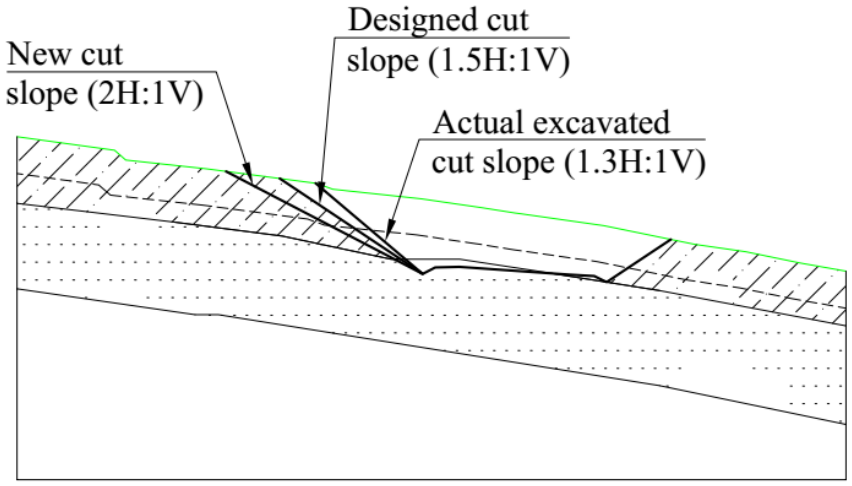


Figure 5. 1: Geometry of slope flattening for station 2+550

Table 5. 1: Analysis result from slope flattening

Slope flattening	Actual cut slope (1.3:1)	Design cut slope (1.5:1)	New cut slope (2:1)
FS	0.950	1.049	1.153

It can be noted that the slope with the original slope is on the verge of failure (FS=1.049) and with the slope ratio of 2:1, it is not satisfactory with FS=1.153. In this regard, providing a retaining structure is mandatory. This is due to:

- The slope flattening is not sufficient to stabilize the cut slope in the safest way
- As the failure is associated with the destabilization of the slope in the time of cutting coupled with the increase of shear stresses due to rainfall, spring and surface streams and steepness of the slope. As a result, a rigid structure can accommodate such failures than any other flexible structure like gabions.
- The vicinity of the cut slope is agricultural land which is not allowed to flatten the cut slopes in excess due to right of way constraints. Hence, the provision of retaining structure is more appropriate.

Accordingly, typical remediation with gravity retaining wall is proposed as it is relatively cheap and the masonry sources are available in the nearby of the cut slope. In the modeling of this structure:

- Peak ground acceleration of 0.15g is used to account the seismicity of the area
- Full saturation condition is considered to account the long term instability problem
- The wall is modeled using soil and interface element
- Besides, parameters in Table 5.2 are used

Table 5. 2: Parameters in the analysis of masonry/retaining wall (Bowles, 1997)

Masonry wall parameter	Units	Value
Unit weight	kN/m^3	25
Elasticity modulus	kN/m^2	2.5E+07
Poisson's ratio	-	0.15
Material model	Linear elastic	
Material type	Non-porous	
Granular material		
Unit weight	kN/m^3	18
Elasticity modulus	kN/m^2	8.0E+04
Poisson's ratio	-	0.3
Cohesion	kN/m^2	1
Angle of internal friction	°	40
Material type	Drained	

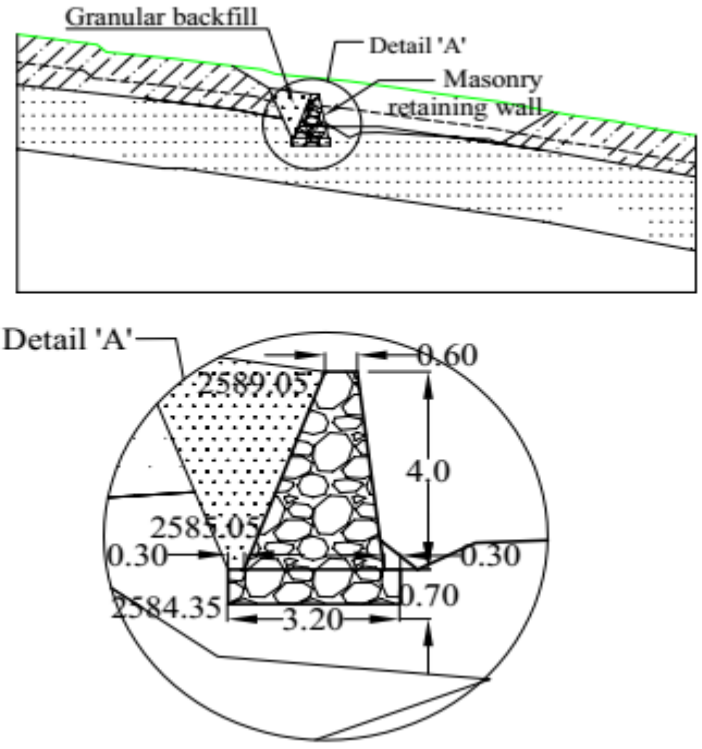


Figure 5. 2: Masonry retaining wall geometry for station 2+550

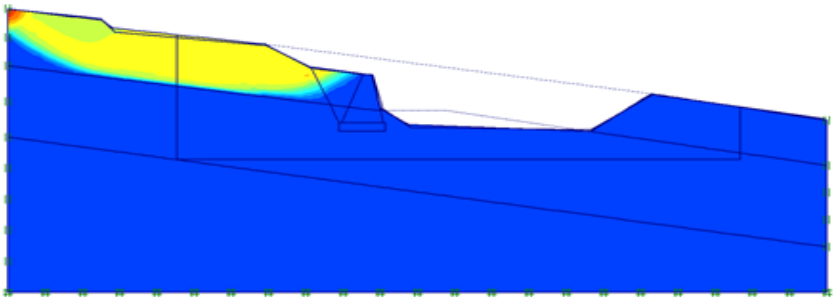


Figure 5. 3: Failure plane after the provision of masonry retaining wall (FS =1.82)

Hence, from this analysis, the FS is improved from 0.984 to 1.82, which has significantly increased the stability of the slope. In addition, the control works (drainage works) can further improve the stability of the slope, which includes;

- Surface drainage: this is to intercept run-off water, surface cut-off drain or furrow ditch should be provided along the crest of the cut slope to prevent water cascading over the new cutting while the batters should be dressed with vegetation to further stabilize.
- Provision of perforated pipes at the place of high concentration of spring water which can guide to the weep holes and other outlet structures.
- Provision of paved longitudinal drainage to collect the water from the cut slope and to prevent the side slope of the road from erosion and infiltration of water.

In the construction of masonry retaining structure, the following activities shall be insured;

- The bottom of the slide shall be excavated after flattening of the slide material during dry weather conditions.
- The foundation should be at an appropriate depth and selected materials like granular or backfill/rock fill materials should be provided.
- Minimize the disturbance due to the removal of toe support of the slide mass during excavation and construction of the wall
- Provide construction joints at least with an interval of 3.5m
- Design the wall which negotiates with the horizontal curvature of the road

- Provide weep holes with an appropriate interval (2m) horizontally and vertically to collect water from the backfill material which eventually discharges it to a longitudinal drainage ditch.

5.2 Station 7+130, RHS

Reasons for failure:

From the result of investigation and analyses, the main reasons for the failure of the cut slope were:

- Originally, the cut slope was designed with a cut slope ratio of 1:1 (H: V) for the lower slope in which the slope is steep for the prevailing materials and the proposed bench was not properly provided yet.
- The presence of springs along the slope and the shallow groundwater table. Further, the occurrence of heavy rainfall after the completion of the excavation of slope to bed road level and this results in excessive saturation of the slope and finally, it leads the flow of slope material.
- The first 6m depth of the slope is composed of elastic silt deposit with some sand and hence, this formation is weak and susceptible to slope failures when exposed to wet.
- The presence of different bedding planes which are the weak plane in residual soil formation for the first layer and relatively competent underlying sandy silt with rock fragments. Accordingly, these beddings plaid a major role in the failure of the cut slope.

Remedial Measures:

This cut slope is highly affected by slope instability and to avert this problem, appropriate remedial measures are needed. In the previous section, slope stability analysis was done based on the design cut slope even though the actual cut slope is not excavated as per the designed cut slope's standard. This is due to the evidence that the design cut slope by itself is unstable for the prevailing conditions. Accordingly, typical remediation with a masonry retaining wall with different arrangement is proposed. In the modeling of this structure, the input conditions listed in station 2+550 are implemented here too.

When one masonry wall is provided (detail 'A' in Figure 5.4 below), the stability of the slope is not that much satisfactory (FS=1.34) and when two masonry walls are provided (in combination of detail 'A' and 'B' in Figure 5.4), the stability of the slope satisfactory (FS=1.63). The reason for the provision of masonry wall B (detail 'B') is that the upper layer of the slope is weak and the layer immediately below this weak soil is relatively competent and hence, retaining structure is necessary at the interface of these soil layers to avert the slides from the upper weak soil.

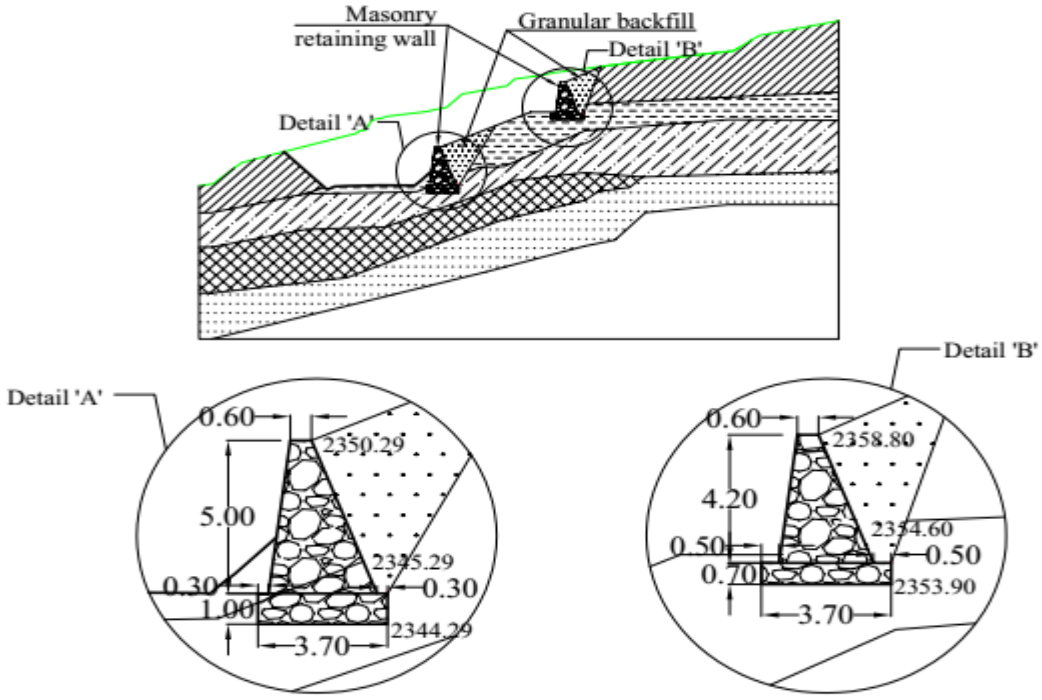


Figure 5. 4: Two masonry retaining walls on the RHS of the road for station 7+130

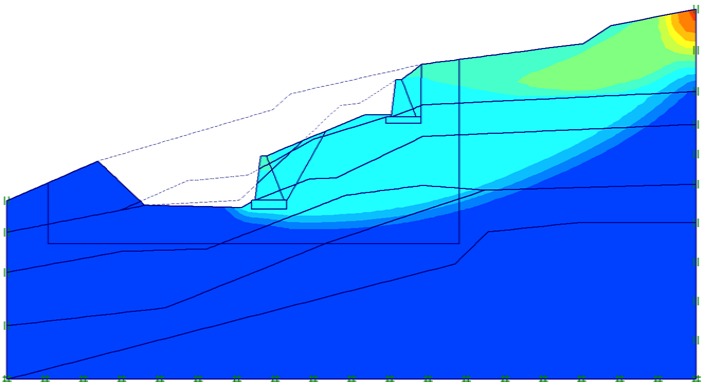


Figure 5. 5: Analysis result after the provision of two masonry retaining walls (FS =1.63)

Implementing a combination of two masonry retaining walls is relatively a good remedial method for this station. In addition, the under listed methods can further improve the stability of the slope.

- Remove a portion of the sliding soil materials from the top part of the slide section to reduce the driving force.
- A surface drainage system, cut-off drain, or paved furrow ditch, should be provided to intercept run-off water along the crest of cuts that prevent the water cascading over the new cutting.
- Controlling the spring water and providing proper crossing structure for any of the water that flows to the road.

In the construction of the masonry retaining structure in this station, all precautions that are mentioned at station 2+550 should be implemented.

5.3 Station 12+470, RHS

Reasons for failure:

From the result of the investigation, the main reasons for the failure of the cut slope were:

- The relative steepness of the actual cut slope for the prevailing materials and it has reduced the stability of the slope.
- The occurrence of an unusual rainfall after the completion of cut slope construction. And even if groundwater was not encountered within the drilled depth, the slope soil is sensitive to infiltration and has a significant capacity of holding water for a long time.
- The upper formation of the soil is weak and susceptible to failures especially when exposed to surface water.

Remedial Measures:

The effective remedial measures which shall be applied in this cut slope location are the control works and mainly the restraint works. The effects of slope flattening and retaining structure were checked and the results are shown in Table 5.4. The geometry of slope flattening is shown in Figure 5.6 below.

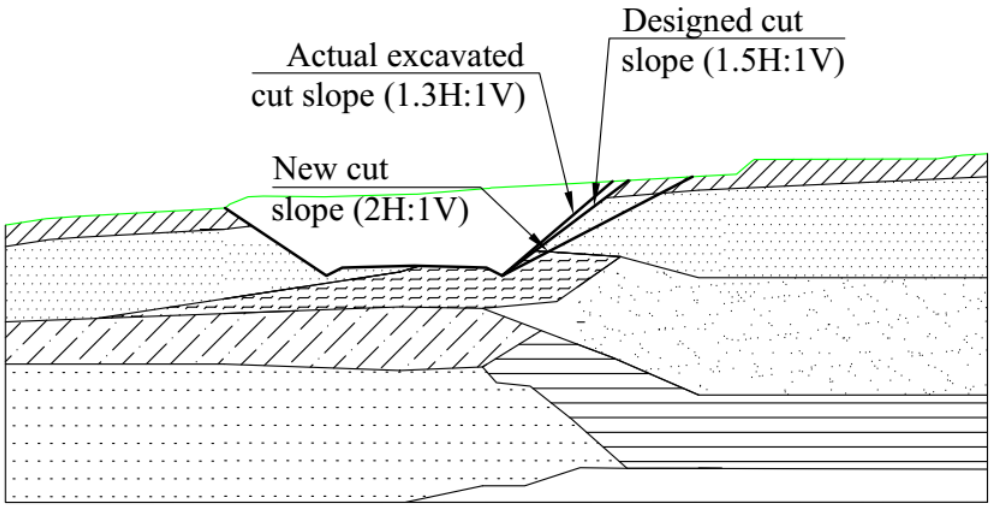


Figure 5. 6: Geometry of slope flattening for station 12+470

The reasons for the provision of retaining wall are the same as the reasons mentioned at station one (2+550) except the increase of shear stresses due to 'high infiltration of surface water'. As a result, a rigid structure can accommodate such failures than any other flexible structure.

Typical remediation with a gravity retaining wall is proposed. In the modeling of this structure, the input conditions listed in station 2+550 are implemented here too.

Table 5. 3: Analysis results of different remedial measures for station 12+470

No.	Remedial Measures	FS
1	Provision of slope ratio of 1.5:1 (H: V)–designed cut slope	1.16
2	Provision of slope ratio of 2:1 (H: V)–new slope flattening	1.21
3	Provision of masonry retaining wall on the RHS	1.41
4	Provision of masonry retaining wall on both RS and LHS	1.86

It can be noted that flattening of the slope alone up to the slope inclination of 2:1 (H: V) is not satisfactory to improve the stability of the cut slope. The provision of retaining structure with an appropriate modification of the slope is necessary. When the slope is provided a masonry wall on RHS only, the stability condition is insufficient and the LHS of the cut slope shows an instability sign (FS=1.41) and when it is provided in both the RHS and LHS, the stability condition is satisfactory (FS=1.86).

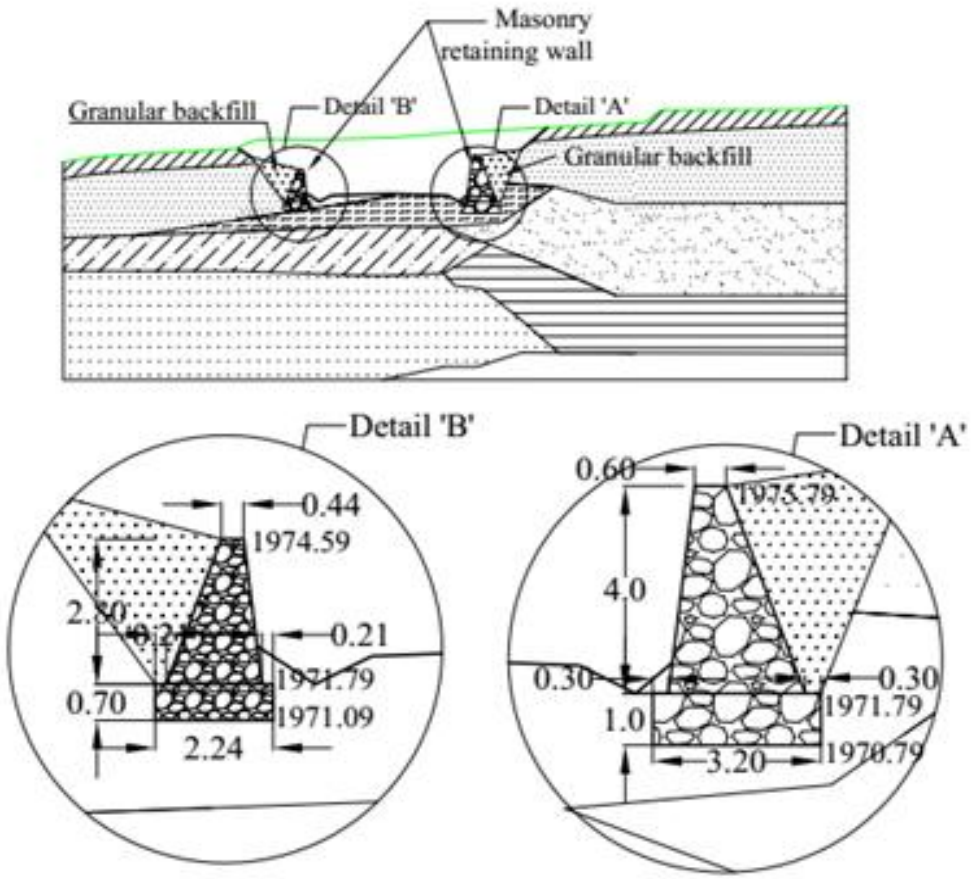


Figure 5. 7: Masonry retaining walls in the RHS and LHS of the road for station 12+470

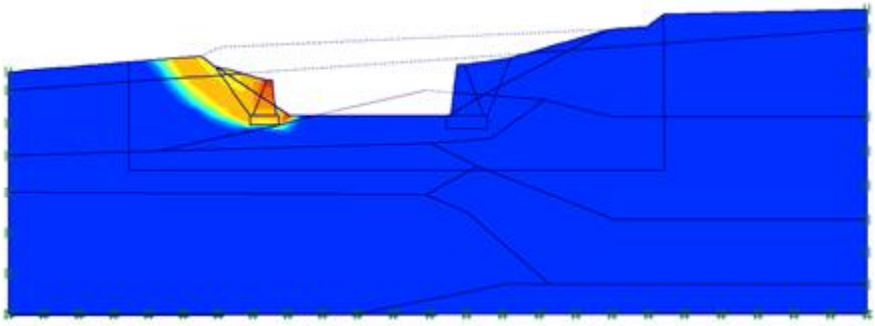


Figure 5. 8: Analysis result after the provision of two masonry retaining walls (FS =1.86)

Therefore, by providing a masonry wall on both sides of the road, it is possible to improve the stability of the cut slope adequately (FS is improved from 1.159 to 1.86). In addition to the above, the control works (drainage works) can further improve the stability of the slope, which includes;

- Provision of paved longitudinal drainage to collect the water from the cut slope and to prevent the side slope of the road from erosion and infiltration of water.
- As much as possible, keeping the slope dry (which is extremely important) and protecting siltation on the top of the retaining wall by providing surface drainage, granular material and vegetation.

In the construction of the masonry retaining structure in this station, all precautions that are mentioned at station 2+550 should be implemented.

6: CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusion

In this study, the investigation, analysis and recommendation of possible remedial measures for the three selected failed cut slope sections located at station 2+550, 7+130 and 12+470 of the Ankober-Dulecha road project are conducted. The 2D finite element analysis method (using PLAXIS software) was employed, and in the remedial solution analyses, the earthquake effect is considered. Furthermore, drained and undrained analyses were conducted and the drained analysis is found critical than undrained analysis; however, for the three cases considered here (undrained, drained and full saturation), full saturation condition is the worst scenario for the subject cut slopes.

It was observed that the main reason for the failure of cut slopes are the provision of a steep cut slope, the presence of groundwater at shallow depth and spring water along the slope, the presence of different residual bedding profiles among the layer of the slope and low shear resistance nature of the cut slope soils.

Depending on the nature of instability incurred, the possible remedial solutions were proposed. Accordingly, control and restraining works or a combination of the two were recommended for each of the investigated slopes to effectively control the problem.

For the three stations, provision of gravity retaining walls with appropriate construction joints and weep holes were recommended. Besides, the following control works are recommended;

- Provision of cut-off drain or furrow ditch along the crest of the cut slope to intercept run-off water and hence, to keep the slope dry and to protect siltation on the top of the retaining wall.
- Provision of paved longitudinal side drainage to collect the water from the cut slope and to prevent the side slope of the road from erosion and infiltration of water.
- Provision of subsurface drainages (perforated pipe along with the spring) for the first and third stations are recommended.

6.2 Recommendations

In this study, some limitations were observed which needs further improvement in the investigation, analysis and remediation of cut slope instability problems;

- In this study, 2D finite element analysis was used as it gives a conservative value of FS and more appropriate for slope design. However, to describe the most real condition of the failed slope, 3D finite element analysis is recommended.
- To describe the lithological cross-section and the detailed characteristics of the failed slope, it needs a large number of drillings along the failed slope in the longitudinal and transverse direction. However, the drilled boreholes conducted here are limited and to solve this difficulty, ideal boreholes are established by considering the characteristics of the outcrops from excavated parts of the slope and neighboring boreholes.
- In the process of extracting and testing of the triaxial CU test, different problems are encountered that shall be avoided in any geotechnical investigations. These include: the samples are extracted from the face of the failed slope which is highly exposed to disturbance; the samples are extracted from the horizontal orientation which is less accurate than the vertical one; at great depths, there were difficulties to extract soil samples with manually; there was an interruption of power during the process of testing which affects the laboratory result and the triaxial CU test equipment is not calibrated recently. Accordingly, to minimize the effect of the above-listed problems, the shear strength parameters are determined from SPT N- values in collaboration with laboratory test values.

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Appendix–A

Laboratory Test Results from EJ–Engineering

For Station 2+550

EJ Engineering
Abiy Tsegaye Construction Related Professional Consultancy Service and Soil Testing
Lafto Sub City, Woreda 10, Tel: +25191003697, Email: abiyt@gmx.de

Project :- Ankober -Awash Arba Junction Road Project ;Contract:I Ankober -Dulecha
Client :- CWCE
Location :- Ankober

1. Soil Classification Summary

No	Station (Km)	Depth (m)	Specific Gravity	Unit Weight (KN/m^3)	UCS (Kpa)	NMC AASHTO T-265	Gradation AASHTO T-88				Atterberg Limit (AASHTO T-89 & T-90)			AASHTO Soil Classification AASHTO (M-145)	Direct shear strength		Unified Soil Classification (ASTM-D2487)
							4.75 mm	2.00 mm	0.425 mm	0.075 mm	LL (%)	PL (%)	PI (%)		C (KN/m^2)	Ø (Degree)	
1	2+040	3.00-3.45	2.61	-	-	38.57	79.40	78.30	71.60	70.30	48.20	25.35	22.85	A-7-6(16)	-	-	CL(Graelly Lean Clay)
2		4.30-4.50	-	16.00	98.00	-	-	-	-	-	-	-	-	-	23.00	20.00	-
3		4.50-4.95	-	-	-	50.20	97.30	96.90	96.60	95.60	58.90	32.37	26.53	A-7-5(20)	-	-	CH(Fat Clay)
4		7.50-7.95	-	16.57	-	-	-	-	-	-	-	-	-	-	25.00	23.00	-
5		9.00-9.21	2.59	-	-	26.18	97.10	96.10	92.40	84.50	52.50	27.04	25.46	A-7-5(20)	-	-	CH(Fat Clay With Sand)
6		10.50-10.63	-	16.92	-	-	-	-	-	-	-	-	-	-	37.00	15.00	-
7		12.00-12.14	-	-	-	25.79	100.00	98.90	92.60	87.30	59.50	27.73	31.77	A-7-5(20)	-	-	CH(Fat Clay)
8	2+070	2.40-3.00	-	16.16	143.00	-	-	-	-	-	-	-	-	-	39.00	16.00	-
9		3.00-3.45	-	-	-	40.47	99.70	99.20	96.80	95.90	60.50	30.93	29.57	A-7-5(20)	-	-	CH(Fat Clay)
10		4.50-4.95	2.53	-	-	43.34	100.00	99.80	97.80	96.80	65.25	36.15	29.10	A-7-5-(20)	-	-	CH(Fat Clay)
11		5.40-6.00	-	17.15	122.00	-	-	-	-	-	-	-	-	-	37.00	20.00	-
12		6.00-6.45	-	-	-	51.18	100.00	100.00	99.80	98.90	56.75	30.32	26.43	A-7-5(20)	-	-	CH(Fat Clay)
13		9.00-9.45	2.61	-	-	33.58	84.30	83.60	83.10	82.20	40.50	20.32	20.18	A-7-6(16)	-	-	CL(Lean Clay With Gravel)
14		10.50-10.52	-	16.08	-	-	-	-	-	-	-	-	-	-	28.00	18.00	-
15		15.00-15.13	-	-	-	33.57	100.00	99.80	82.80	77.40	58.25	30.42	27.83	A-7-5(20)	-	-	CH(Fat Clay With Sand)
16		16.50-16.75	-	16.81	-	-	-	-	-	-	-	-	-	-	29.00	19.00	-
17		18.00-18.07	-	17.13	-	28.34	100.00	100.00	97.20	94.50	58.25	33.65	24.60	A-7-5(20)	-	-	CH(Fat Clay)
18	2+550	1.50-1.95	-	-	-	24.01	58.50	54.70	48.80	46.40	38.50	20.26	18.24	A-6(5)	-	-	GC(Clayey Gravel)
19		3.00-3.45	-	17.15	-	-	-	-	-	-	-	-	-	-	22.00	18.00	-
20		4.50-4.95	-	-	-	43.31	99.20	98.90	98.70	98.00	68.67	36.71	31.96	A-7-5(20)	-	-	CH(Fat Clay)
21		5.30-5.50	-	16.49	157.00	-	-	-	-	-	-	-	-	-	40.00	24.00	-
22		6.00-6.45	2.64	-	-	42.23	99.00	96.30	93.50	90.20	52.96	28.20	24.76	A-7-6(20)	-	-	CH(Fat Clay)
23		7.50-7.95	-	16.57	-	-	-	-	-	-	-	-	-	-	20.00	14.00	-
24		9.00-9.45	-	-	-	45.65	99.30	96.00	85.10	78.90	59.12	29.69	29.43	A-7-6(20)	-	-	CH(Fat Clay With Sand)

Tested by :- Kassay Argaw
Date :- 04/06/2018
Signiture :- _____

Approved By :- Samrawit Silishi
Date :- 04/06/2018
Signiture :- _____

For Station 7+130

EJ Engineering

Abiy Tsegaye Construction Related Professional Consultancy Service and Soil Testing

Lafto Sub City, Woreda 10, Tel: +251910008BT, Email: abjet@gmx.de

Project :-Ankober -Awash Arba Junction Road Project Contract -I: Ankober -Dulecha
Client :- CWCE
Location :-Ankober

Summary For Soil & Classification

No	Station (Km)	Depth (m)		Unit Weight (KN/m^3)	Specific Gravity	NMC AASHTO (T-265)	Gradation AASHTO T-88				Atterberg Limit (AASHTO T-89 & T-90)			Direct shear strength		AASHTO Soil Classification (AASHTO M-145)	Unified Soil Classification (ASTM-D2487)
		Top	Base				4.75 mm	2.00 mm	0.425 mm	0.075 mm	LL (%)	PL (%)	PI (%)	C (KN/m^2)	ø (Degree)		
23	7+130	2.20	2.40	16.76	-	-	-	-	-	-	-	-	-	40.00	24.00	-	-
24		3.00	3.45	-	2.62	53.98	95.90	92.70	82.40	73.00	53.94	35.46	18.48	-	-	-	MH(Elastic Silt with Sand)
25		4.50	4.95	17.40	-	-	-	-	-	-	-	-	-	48.00	16.00	-	-
26		5.40	5.60	17.11	2.55	-	-	-	-	-	-	-	-	43.00	13.00	-	-
27		6.00	6.45	-	-	47.00	86.80	84.20	65.90	50.10	40.50	24.68	15.82	-	-	A-6(3)	CL(Sandy Lean Clay)
28		7.50	7.78	-	2.60	24.95	100.00	100.00	81.00	67.30	40.50	21.66	18.84	-	-	A-7-6(11)	CL(Sandy Lean Clay)
29		9.00	9.18	17.12	-	-	-	-	-	-	-	-	-	38.00	20.00	-	-
30		10.50	10.78	-	2.64	29.56	99.30	98.60	90.60	87.10	52.17	33.88	18.29	-	-	A-7-5(20)	CL(Lean Clay)
31		12.00	12.08	-	-	34.48	100.00	100.00	64.70	48.00	43.12	29.92	13.20	-	-	A-7-6(4)	-
32		13.50	13.55	17.63	-	-	-	-	-	-	-	-	-	35.00	17.00	-	-
33		15.00	15.13	-	-	21.59	98.30	96.90	91.90	86.90	47.00	27.71	22.29	-	-	A-7-6(21)	CL(Lean Clay)
34		18.00	18.13	-	2.56	26.76	99.80	90.70	71.80	53.60	46.42	32.86	13.56	-	-	A-7-5(6)	CL(Sandy Lean Clay)

Tested By :- Gizew Ezezew :———— **Approved By:-Samrawit Silishi :**————

For Station 7+210

EJ Engineering

Aby Tsegaye Construction Related Professional Consultancy Service and Soil Testing

Lafto Sub City, Woreda 10, Tel: +251910030697, Email: abyt@gmx.de

Project :-Ankober -Awash Arba Junction Road Project

Client :- CWCE

Summary For Soil & Classification

No	Station (Km)	Depth (m)		Unit Weight (KN/m³)	Specific Gravity	NMC AASHTO (T-265)	Gradation AASHTO T-88				Atterberg Limit (AASHTO T-89 & T-90)			Direct shear strength		AASHTO Soil Classification (AASHTO M-145)	nified Soil Classification (ASTM-D248
		Top	Base				4.75 mm	2.00 mm	0.425 mm	0.075 mm	LL (%)	PL (%)	PI (%)	C (KN/m²)	Ø (Degree)		
35	7+210	1.50	1.95	-	2.52	41.48	82.80	81.70	71.00	62.30	52.75	36.15	16.60	-	-	A-7-5(10)	MH(Sandy Elastic Silt With Gravel)
36		3.00	3.45	17.37	-	-	-	-	-	-	-	-	-	39.00	15.00	-	-
37		4.50	4.95	-	2.43	36.73	100.00	100.00	67.30	46.70	40.00	27.35	12.65	-	-	A-6(3)	SM(Silty Sand)
38		5.00	5.20	17.95	-	-	-	-	-	-	-	-	-	29.00	19.00	-	-
39		6.00	6.12	-	2.45	36.73	100.00	100.00	79.10	56.80	43.00	26.91	16.09			A-7-6(7)	CL(Sandy Lean Clay)
40		7.50	7.70	18.81	-	-	-	-	-	-	-	-	-	14.00	20.00	-	-
41		9.00	9.12	-	-	27.58	85.00	81.40	54.20	40.30	NP	NP	NP	-	-	A-4(0)	ML(Silty Sand)
42		12.00	12.13	17.96	2.49	27.29	99.60	98.10	75.90	57.90	NP	NP	NP	17.00	27.00	A-4(0)	ML(Sandy Silt With Gravel)
43		13.50	13.63	18.16	-	-	-	-	-	-	-	-	-	12.00	22.00	-	-
44		15.00	15.11	-	2.46	33.02	100.00	98.30	90.10	61.10	NP	NP	NP	-	-	A-4(0)	ML(Sandy Silt)
45		18.00	18.10	17.89	-	34.11	92.40	91.30	84.90	80.70	46.64	32.15	14.49	-	-	A-7-5(14)	CL(Sandy Lean Clay With Sa

Tested By > Gizew Ezezew :---------- Approved By>Samrawit Silishi :----------

For Station 12+470 and 12+490

EJ Engineering
Abiy Tsegaye Construction Related Professional Consultancy Service and Soil Testing
Lafto Sub City, Woreda 10, Tel: +25191003697, Email: abiyt@gmx.de

Project :- Ankober -Awash Arba Junction Road Project ;Contract:I Ankober -Dulecha
Client :- CWCE
Location :- Ankober

1. Soil Classification Summary

No	Staion	Depth	Specific gravity	Unit Weight (KN/m³)	UCS(Kpa)	NMC AASHTO (T-265)	Gradation AASHTO T-88				Atterberg Limit (AASHTO T-89 & T-90)			Direct shear strength		AASHTO Soil Classification AASHTO (M-145)	Unified Soil Classification (ASTM-D2487)
							4.75 mm	2.00 mm	0.425 mm	0.075 mm	LL (%)	PL (%)	PI (%)	C (KN/m²)	Ø (Degree)		
25	12+470	2.00-2.20	-	17.24	109.00	-	-	-	-	-	-	-	-	23.00	21.00	-	-
26		3.00-345	2.64	-	-	28.67	61.40	58.40	55.00	52.50	54.25	28.53	25.72	-	-	A-7-6(11)	CH(Gravelly Fat Clay)
27		6.00-6.45	-	-	-	37.71	90.80	86.00	85.00	82.50	47.25	24.83	22.42	-	-	A-7-6(20)	CL(Lean Clay With Gravel)
28		9.00-9.45	2.60	-	-	28.46	95.00	90.90	90.00	88.20	55.60	29.17	26.43	-	-	A-7-6(20)	CH(Fat Clay)
29		11.00-11.40	-	17.63	143.00	-	-	-	-	-	-	-	-	42.00	15.00	-	-
30		12.50-12.95	-	-	-	35.00	94.50	91.90	91.40	90.00	72.25	37.78	34.47	-	-	A-7-5(20)	CH(Fat Clay)
31		15.50-15.95	2.56	-	-	35.89	95.90	94.40	94.10	92.90	73.44	36.85	36.59	-	-	A-7-5(20)	CH(Fat Clay)
32		19.00-19.45	-	16.14	-	40.53	99.70	98.80	98.50	90.70	50.00	26.39	23.61	43.00	12.00	A-7-6(20)	CH(Fat Clay)
33	12+490	1.50-1.95	-	-	-	32.45	94.90	91.80	85.00	83.90	51.25	27.22	24.03	-	-	A-7-6(20)	CH(Fat Clay With Sand)
34		3.80-4.00	-	16.99	108.00	-	-	-	-	-	-	-	-	26.00	16.00	-	-
35		4.50-4.95	2.60	-	-	23.22	76.90	70.70	64.50	62.90	42.50	23.26	19.24	-	-	A-7-6(10)	CL(Gravelly Lean Clay)
36		6.00-6.45	-	16.95	-	25.00	75.80	71.10	70.50	68.70	46.50	23.84	22.66	40.14	14.00	A-7-6(15)	CL(Gravelly Lean Clay)
37		8.50-8.95	2.55	-	-	32.18	98.00	96.70	96.10	94.70	70.50	24.55	45.95	-	-	A-7-6(20)	CH(Fat Clay)
38		10.00-10.45	-	-	-	37.57	96.30	94.70	94.30	92.70	56.00	29.01	26.99	-	-	A-7-6(20)	CH(Fat Clay)
39		11.00-11.40	-	17.00	132.00	-	-	-	-	-	-	-	-	33.00	29.00	-	-
40		12.50-12.65	2.58	-	-	15.52	85.30	81.10	73.60	68.10	38.50	19.10	19.40	-	-	A-7-6(11)	CL(Sandy Lean Clay With Gravel)
41		16.50-16.70	-	17.08	-	-	-	-	-	-	-	-	-	24.00	19.00	-	-
42		18.50-18.60	-	-	-	14.29	87.40	80.20	78.80	75.90	46.50	23.84	22.66	-	-	A-7-6(17)	CL(Lean Clay With Gravel)

Tested BY :-Kassay Argaw
Date :-04/09/2018
Signature:----------

Approved By :-Samrawit Silishi
Date :- 04/09/2018
Signature:----------

Appendix–B

Triaxial Shear (CU) Test Results by the Researcher

For station 2+550

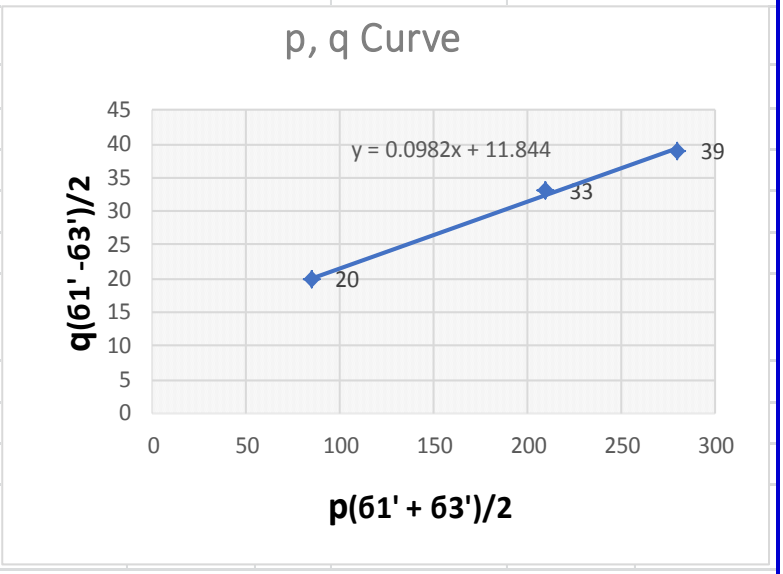
	Project:	Ankober–Dulecha	Sample type:	Undisturbed
	Client:	Yetsedaw	Test type:	Triaxial CU test
	Location:	2+550	Date:	November 16,2019

a Depth: 3.5 m

P	q
(61' + 63')/2	(61' -63')/2
85	20
210	33
280	39

$\emptyset' = \sin^{-1} 0.0982 = 5.63^{\circ}$

$C' = \frac{11.84}{\cos 5.63} = 11.89\ kPa$



b Depth: 6.8 m

P	q
(61' + 63')/2	(61' -63')/2
72	42
152	49
275	62

Φ'=5.69°

C'=34.68 kPa

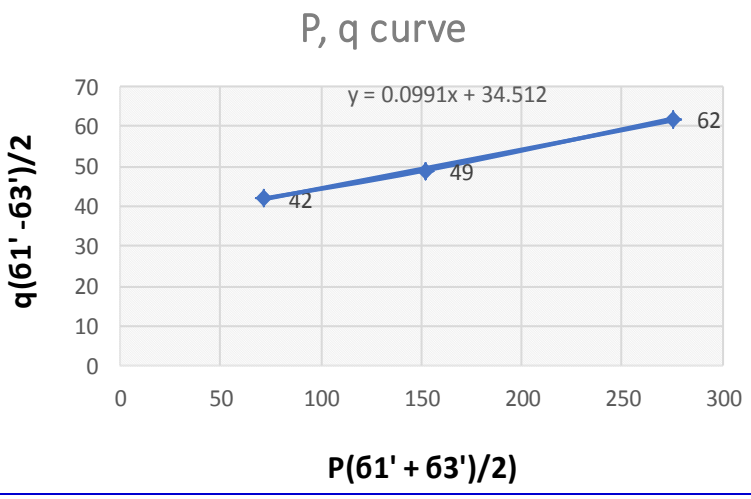


Figure B–1: Effective shear strength parameters at station 2+550

Deviatorin Stress Versus Strain Curves at station 2+550

a) At 3.5m depth:

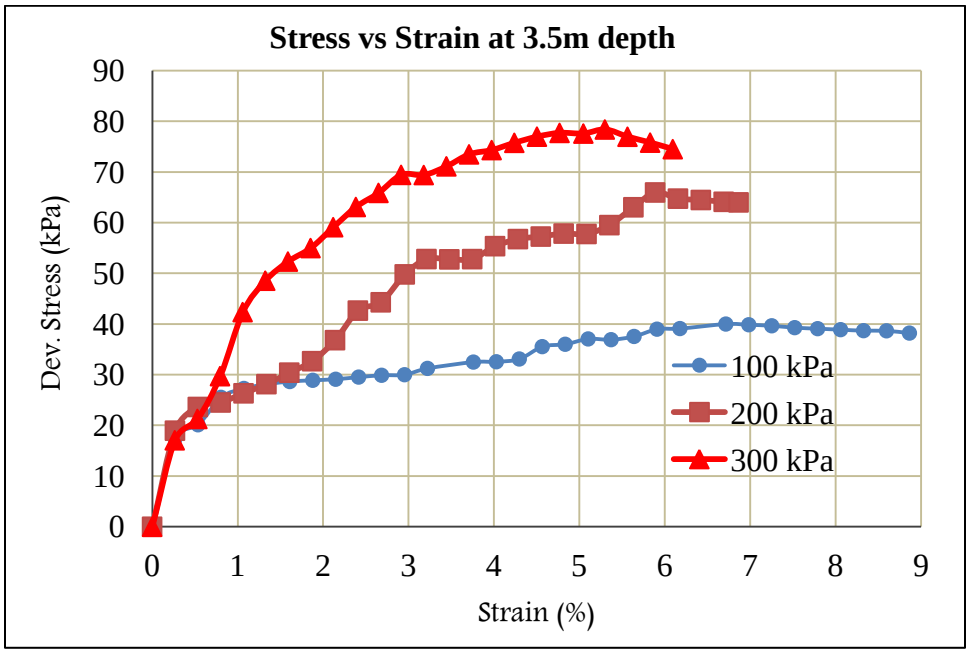


Figure B–2: Stress–strain curves at 3.5m depth

b) At 6.8m depth:

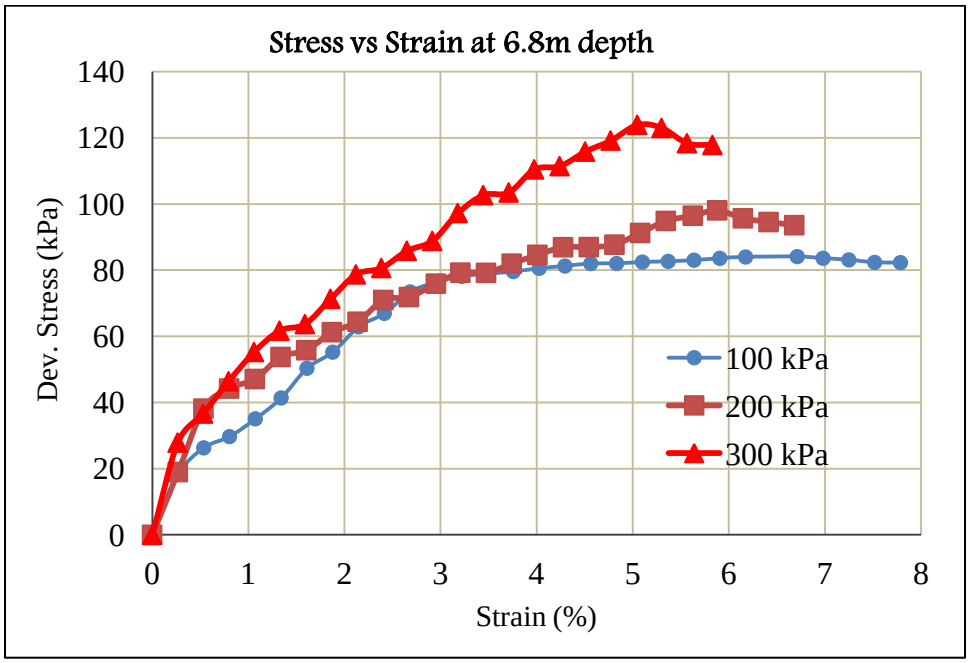


Figure B–3: Stress–strain curves at 6.8m depth

For station 7+130

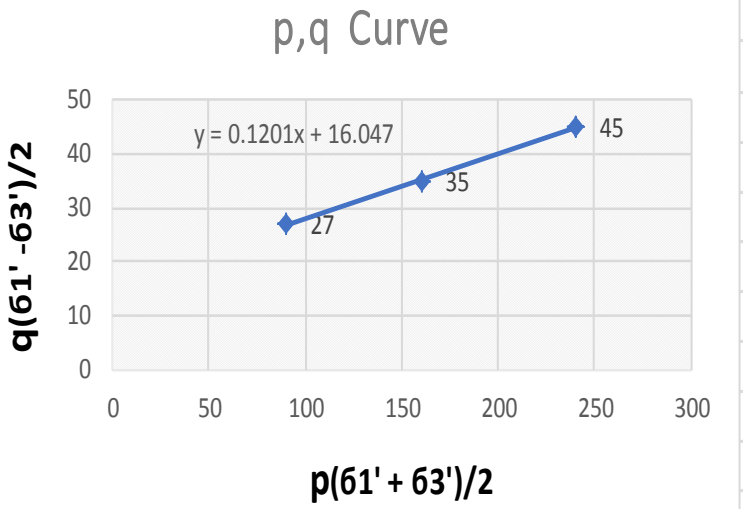
	Project:	Ankober-Dulecha	Sample type:	Undisturbed
	Client:	Yetsedaw	Test type:	Triaxial CU test
	Location:	7+130	Date:	December 28, 2019

a Depth: 3.7 m

P	q
(61' + 63')/2	(61' -63')/2
90	27
160	35
240	45

C'=16.14 Kpa

Φ'=6.9°



b Depth: 8.5m

P	q
(61' + 63')/2	(61' -63')/2
82	49
195	68
334	95

C'=33.97Kpa

Φ'=10.54°

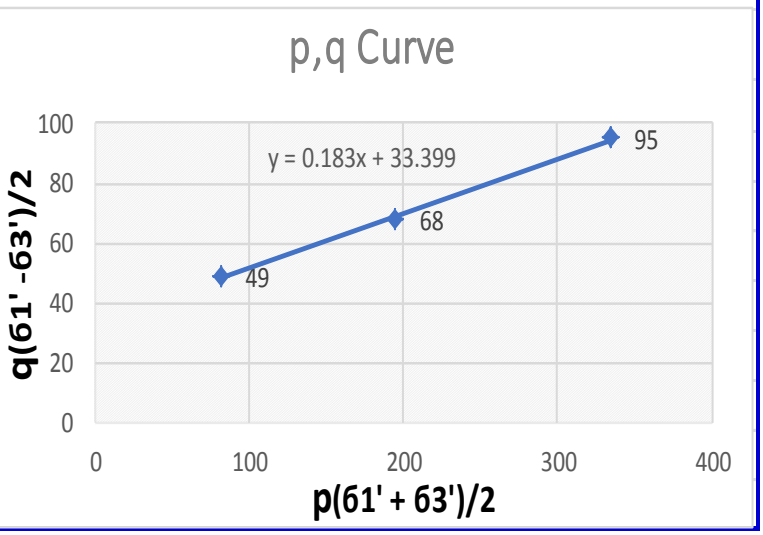


Figure B–4: Effective shear strength parameters at station 7+130

Deviatoric Stress Versus Strain Curves at station 7+130

a) At 3.7m depth:

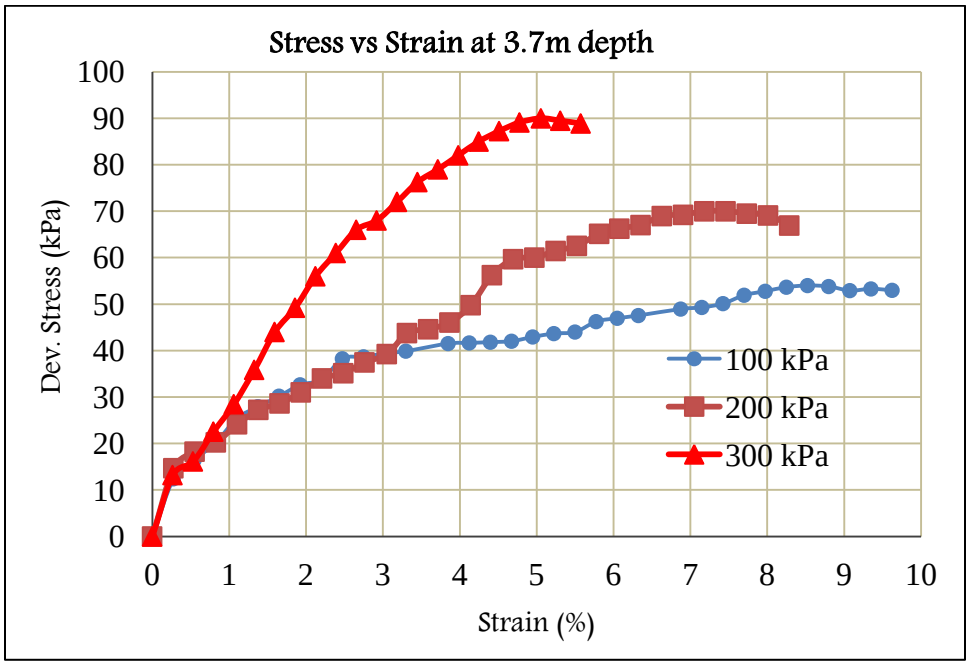


Figure B-5: Stress-strain curves at 3.7m depth

b) At 8.5m depth:

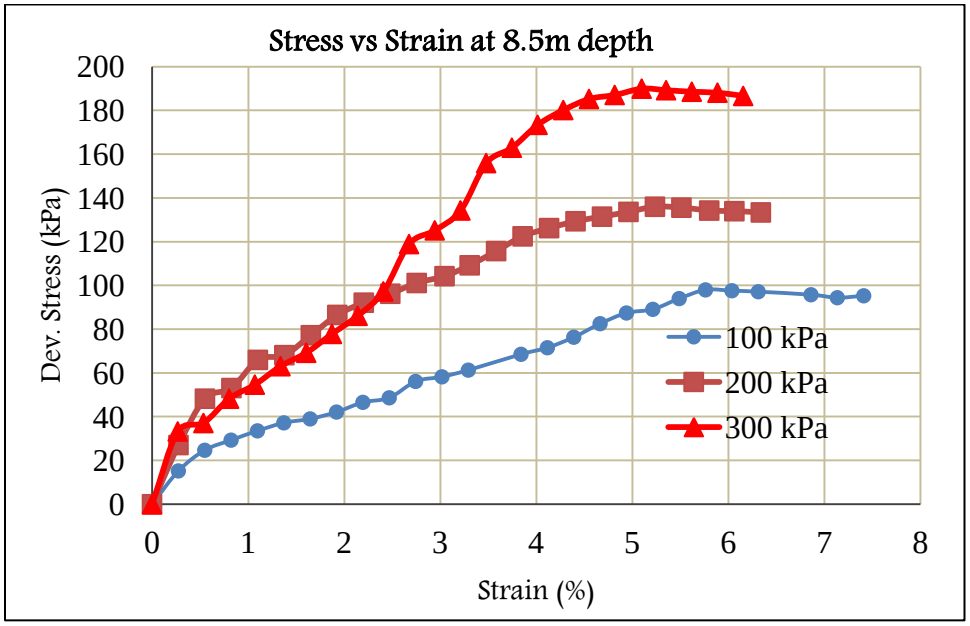


Figure B-6: Stress-strain curves at 8.5m depth

For station 12+470

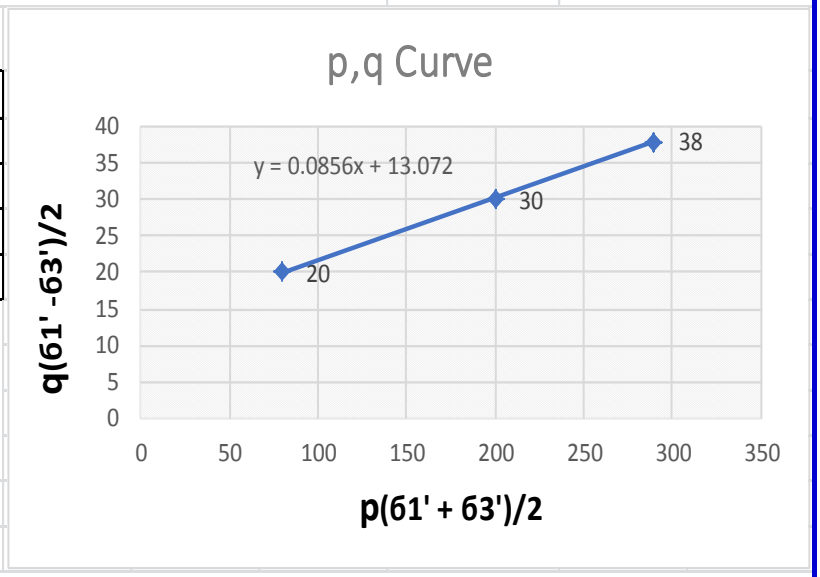
	Project:	Ankober-Dulecha	Sample type:	Undisturbed
	Client:	Yetsedaw	Test type:	Triaxial CU test
	Location:	12+470	Date:	February 02,2020

a Depth: 3.2m

P	q
(61' + 63')/2	(61' -63')/2
80	20
200	30
290	38

C'=13.12 KPa

Φ'=4.91˚



b Depth: 6.5m

P	q
61' + 63')/2	(61' -63')/2
150	55
250	68
300	77

C'=33.35 KPa

Φ'=8.3˚

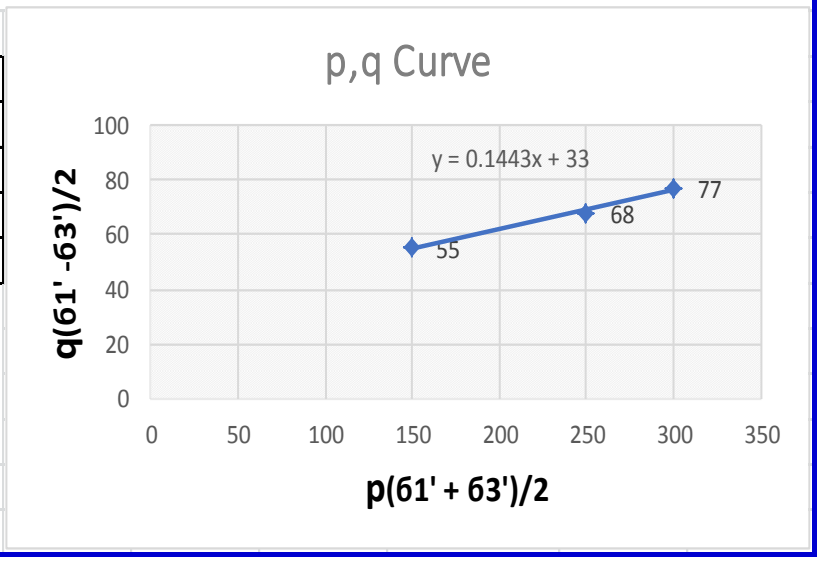


Figure B–7: Effective shear strength parameters at station 7+130

Deviatoric Stress Versus Strain Curves at station 12+470

a) At 3.2m depth:

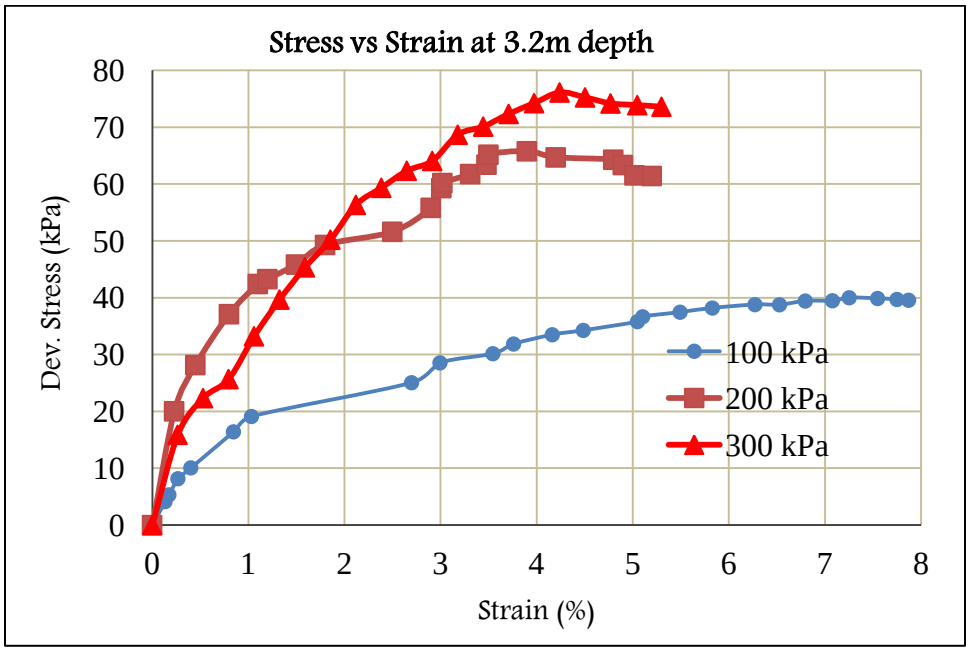


Figure B-8: Stress-strain curves at 3.2m depth

b) At 6.5m depth:

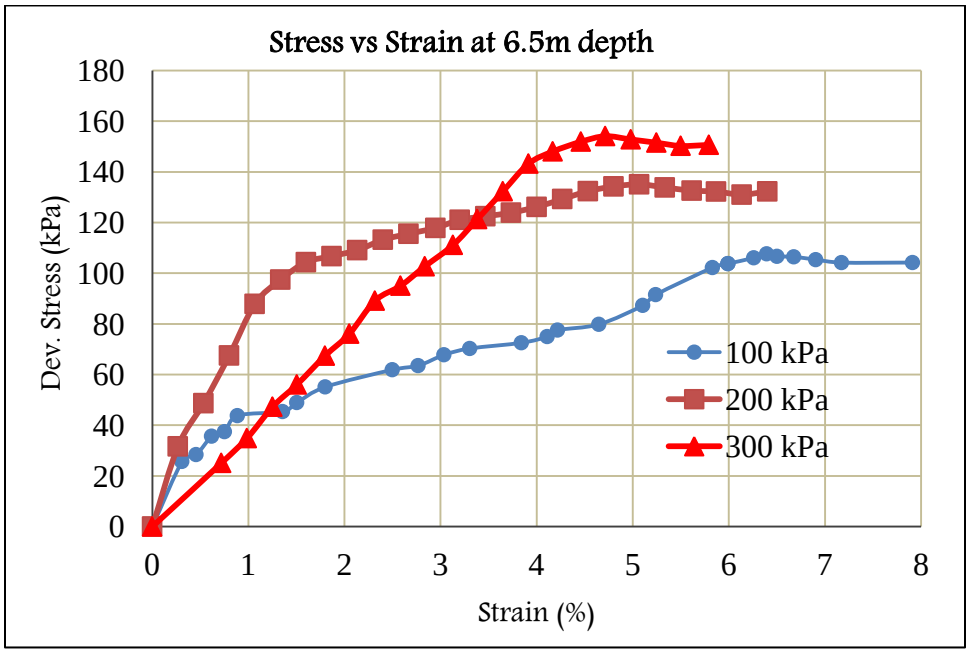


Figure B-9: Stress-strain curves at 6.5m depth

APPENDIX–C

Determination of Modulus of Elasticity (E &E')

	For station 2+550										
					$N_{corr(60)}=E_h*C_b*C_s*C_r*N/0.6$	$N_{30}=N_{corr(60)}*(60/30)$	$\mu=4*N_{30}+50$	$E=\mu*Pa((q_z+0.5\Delta q_z)/Pa)^{(w)}$		$E'_{eq}=2(1+v')*E_{eq}/3$, v'=0.35	
Layer No.	z	γ	σ_z	N	$N_{corr(60)}$	N_{30}	μ	E_i	E_{eq}	E'_{eq}	GWT
1	1.95	17.15	33.44	5	4	8.75	85	4,430.10	6,389.82	5,750.84	2.5
	3.45	17.15	49.25	11	10	19	127	8,349.54			
2	4.95	16.53	59.33	21	21	41.65	217	15,923.49	18,629.96	16,766.96	
	6.45	17.54	69.41	17	19	38	201	16,213.92			
	7.95	17.5	79.49	16	18	35	192	16,811.34			
	9.45	17.4	89.57	25	28	55	272	25,571.08			
	For station 7+130										
1	1.95	17.1	33.44	11	10	19	127	6,618.79	8,769.87	7,892.88	3.5
	3.45	17.1	58.99	6	5	11	92	6,740.16			
	4.95	17.1	70.01	12	12	24	145	11,789.02			
	6.45	17.38	81.01	7	8	16	112	9,931.50			
2	7.78	17.38	94.48	50	55	111	493	47,946.79	49,515.93	44,564.34	
	9.18	17.38	105.01	50	55	111	493	51,085.08			
3	10.78	17.38	117.21	50	58	117	517	57,148.45	61,628.33	55,465.50	
	12.08	17.38	127.05	50	58	117	517	59,980.55			
	13.55	17.38	138.18	50	58	117	517	63,080.15			
	15.13	17.38	150.15	50	58	117	517	66,304.17			
4	16.64	17.38	161.56	50	58	117	517	69,282.88	72,753.63	65,478.27	
	18.13	17.38	172.84	50	58	117	517	72,145.99			
	19.09	17.38	180.11	50	58	117	517	73,951.73			
	20	17.38	186.99	50	58	117	517	75,633.93			
	For station 7+210										
1	1.95	17.37	33.8715	4	4	7	78	4,096.48	7,003.44	6,303.10	3.5
	3.45	17.37	59.9265	12	11	21	134	9,910.40			
2	4.63	17.95	69.53	50	50	99	447	36,116.20	40,184.10	36,165.69	
	6.12	18.81	82.66	50	55	111	493	44,252.00			
3	7.7	18.81	96.88	50	55	111	493	48,673.90	50,552.23	45,497.01	
	9.12	18.81	109.66	50	55	111	493	52,430.57			
4	10.62	17.96	121.89	50	58	117	517	58,506.82	60,245.23	54,220.71	
	12.13	17.96	134.2	50	58	117	517	61,983.65			
5	13.63	18.16	146.73	50	58	117	517	65,393.86	68,637.02	61,773.32	
	16.63	18.16	171.78	50	58	117	517	71,880.18			
6	20	17.89	199.1	50	58	117	517	78,535.93	78,535.93	70,682.34	

For station 12+470											
	2.2	17.24	37.93							-	
1	3.45	17.24	59.48	7	6	12	99	7,289.08			
	4.95	17.24	85.34	12	12	24	145	13,276.19	12,125.53	10,912.98	
	6.45	17.24	111.2	11	12	24	148	15,811.32			
2	7.95	17.24	137.1	24	27	53	263	31,934.71			
	9.45	16.91	162.6	25	28	55	272	36,569.91			
	11.4	16.91	195.57	21	25	49	246	36,994.03	38,738.35	34,864.52	Nil
	12.95	16.91	221.78	23	25	51	254	41,180.19			
	14.45	16.91	247.14	25	28	55	272	47,012.93			
3	15.5	16.91	264.89	32	35	71	334	60,207.99			
	19.45	16.91	331.68	26	29	58	281	57,920.36	59,005.91	53,105.32	
	20	16.91	340.98	26	29	58	281	58,889.38			
For station 12+490											
1	1.95	16.97	33.0915	7	6	12	99	5,127.20	8,147.73	7,332.96	
	3.45	16.97	58.5465	13	13	26	153	11,168.26			
2	4.8	16.97	81.456	38	42	84	387	34,403.70		33,966.21	
	6.45	16.97	109.4565	38	42	84	387	41,076.77	37,740.23		
3	8.95	16.97	151.8815	55	61	122	538	69,475.43			Nil
	10.45	17	177.3815	51	60	119	526	74,601.22	75,422.52	67,880.26	
	12.65	17	214.7815	50	58	117	517	82,190.90			
4	18.6	17.1	316.4815	50	58	117	517	103,713.34	106,032.53	95,429.28	
	20	17.1	340.4215	50	58	117	517	108,351.72			

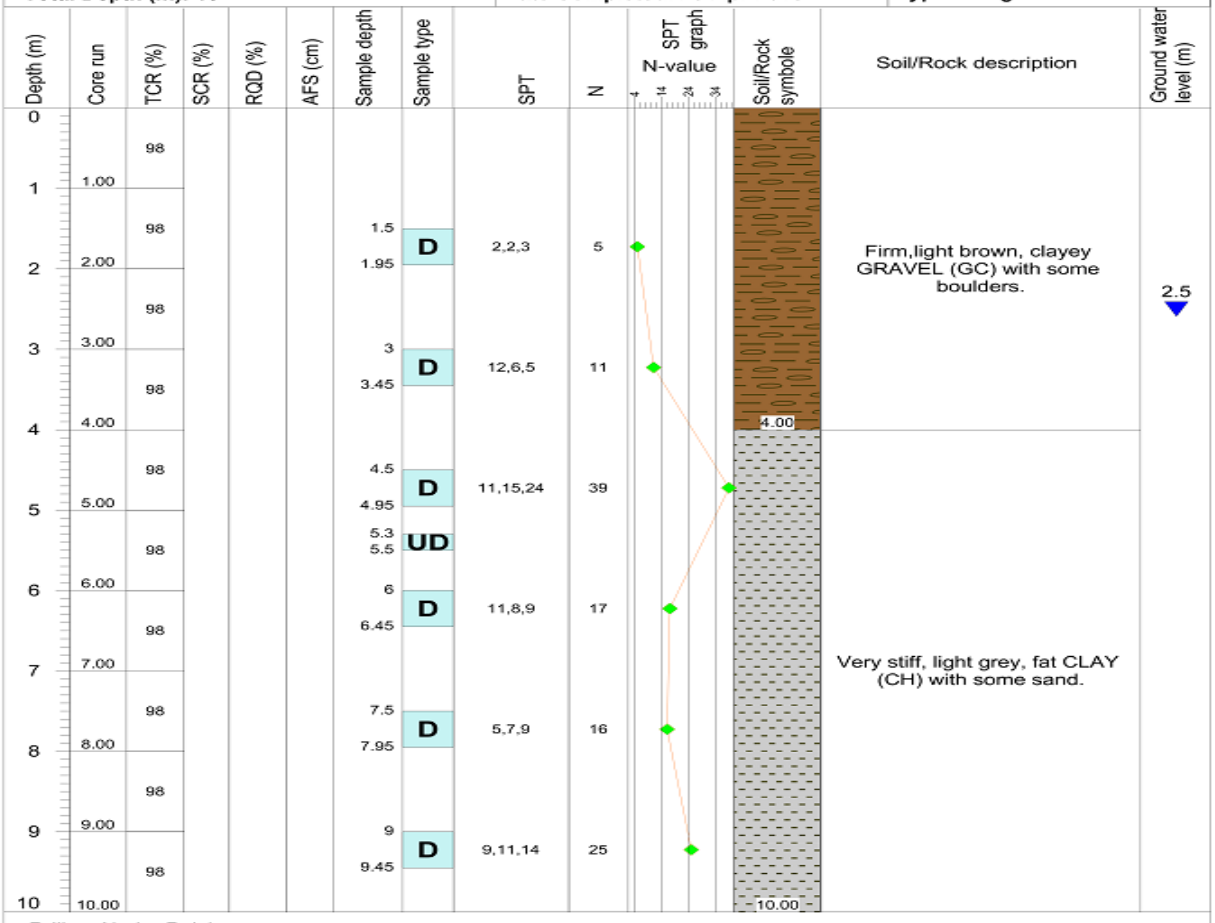
APPENDIX–D

Bore Hole Diagram

For Station 2+550

Project	Contractor	Sub Contractor	Consultant
Ankober-Awash Arba Junction Road Project; Contract-I: Ankober-Dulecha	Sunshine Construction PLC	CWCE	C-Tech Engineering PLC

Location/Chainage: Km 2+550	Drilling Method: Dry boring by single core barrel (0-10m)	
Coordinates (E/N): 580838 1059151	Core diameter (mm): 90mm(0-4.00m), 75 mm (4.0 -10m)	Sheet: 1 of 1
Ground Elevation (masl.): 2592	Casing type & depth: 110mm	BH Inclination: 90°
Borehole No.: 2+550	Date Started: 23-Apr-2018	Bit type: Carbide
Total Depth (m): 10	Date Completed: 25-Apr-2018	Type of Rig: Conventional



Sample depth	Sample type	SPT	N
1.5–1.95	D	2,2,3	5
3–3.45	D	12,6,5	11
4.5–4.95	D	11,15,24	39
5.3–5.5	UD		
6–6.45	D	11,8,9	17
7.5–7.95	D	5,7,9	16
9–9.45	D	9,11,14	25

Driller: Abebe Balcha Logged by: Hagos K/Mariam Checked by: Kassahun Zergaw

Note:
D=Disturbed Sample (Split spoon)
UD=Undisturbed Sample
RC=Rock Core Sample
SPT=Standard Penetration Test

SCR=Solid Core Recovery
RQD=Rock Quality Designation
AFS=Average Fracture Spacing

For Station 7+130

Project	Contractor	Sub Contractor	Consultant
Ankober-Awash Arba Junction Road Project; Contract-I: Ankober-Dulecha	Sunshine Construction PLC	CWCE	C-Tech Engineering PLC

Location/Chainage: Km 7+130	Drilling Method: Dry boring by single core barrel (0-20m)	
Coordinates (E/N): 583020.7 1057887.99	Core diameter (mm): 90mm(0-7.50m), 75 mm (7.5 -20m)	Sheet: 1 of 1
Ground Elevation (masl.): 2362.56	Casing type & depth: 110mm	BH Inclination: 90°
Borehole No.: 7+130	Date Started: 29-Mar-2018	Bit type: Carbide
Total Depth (m): 20	Date Completed: 1-Apr-2018	Type of Rig: Wire line drilling

Depth (m)	Core run	TCR (%)	SCR (%)	RQD (%)	AFS (cm)	Sample depth	Sample type	SPT	N	Soil/Rock symbole	Soil/Rock description	Ground water level (m)
0–1	1.00	98				1.5	D	2,3,10	13		Firm to stiff, dark grey, Elastic SILT with sand (MH) and gravels.	3.5
1–2	2.00	98				1.95						
2–3	3.00	98				2.2 2.4 3	D	2,3,3	6			
3–4	4.00	98				3.45						
4–5	5.00	98				4.5	D	3,5,8	13			
5–6	6.00	98				4.95 5.4 5.6				6.00		
6–7	7.00	98				6	D	6,4,3	7		Very stiff, brownish grey, sandy lean CLAY(CL), Completley weathered product of Ignimbrite & some Tuff.	
7–8	8.00	98				6.45 7.5	D	23,50/R13cm	50			
8–9	9.00	98				7.78				9.00		
9–10	10.00	98				9	D	28,50/R3cm	50		Very stiff, pinkish to light brown, lean CLAY (CL),highly weathered product of Ignimbrite.	
10–11	11.00	98				9.18 10.5	D	38,50/R13cm	50			
11–12	12.00	98				10.78						
12–13	13.00	98				12	D	50/R8cm	50			
13–14	14.00	98				12.08 13.5	D	50/R5cm	50			
14–15	15.00	98				13.55						
15–16	16.00	98				15	D	50/R13cm	50	16.00		
16–17	17.00	98				15.13 16.5	D	50/R14cm	50		Very stiff, light brown, sandy lean CLAY(CL), highly weathered product of Ignimbrite.	
17–18	18.00	98				16.64						
18–19	19.00	98				18 18.13	D	50/R13cm	50			
19–20	20.00	98				19 19.09	D	50/R9cm	50	20.00		

SPT graph N-value: 4, 24, 44

Driller: Tarekegn Tesfaye

Logged by: Hagos K/Mariam

Checked by: Kassahun Zergaw

Note:
D=Disturbed Sample (Split spoon)
UD=Undisturbed Sample
RC=Rock Core Sample
SPT=Standard Penetration Test

SCR=Solid Core Recovery
RQD=Rock Quality Designation
AFS=Average Fracture Spacing

For Station 7+210

Project	Contractor	Sub Contractor	Consultant
Ankober-Awash Arba Junction Road Project; Contract-I: Ankober-Dulecha	Sunshine Construction PLC	CWCE	C-Tech Engineering PLC

Location/Chainage: Km 7+210	Drilling Method: Dry boring by single core barrel (0-20m)	
Coordinates (E/N): 583031.867 10578082.249	Core diameter (mm): 90mm(0-4.00m), 75 mm (4.0 -20m)	Sheet: 1 of 1
Ground Elevation (masl.): 2358.254	Casing type & depth: 110mm	BH Inclination: 90°
Borehole No.: 7+210	Date Started: 27-Mar-2018	Bit type: Carbide
Total Depth (m): 20	Date Completed: 1-Apr-2018	Type of Rig: Conventional

Depth (m)	Core run	TCR (%)	SCR (%)	RQD (%)	AFS (cm)	Sample depth	Sample type	SPT	N	Soil/Rock symbole	Soil/Rock description	Ground water level (m)
0–1	1.00	98				1.5 1.95	D	1,1,,3	4	3.50	Soft to stiff, dark grey, sandy elastic SILT with some gravels (MH).	3.5
1–2	2.00	98										
2–3	3.00	98				3 3.45	D	20,7,5	12			
3–4	4.00	98										
4–5	5.00	98				4.5 4.63	D	50/R13cm	50	6.00	Very stiff, light yellow , silty SAND (SM) ,Completley weathered product of Tuff.	
5–6	6.00	98				5 5.2						
6–7	7.00	98				6 6.12	D	50/R12cm	50	9.00	Very stiff, pinkish brown, sandy lean CLAY (CL) ,highly weathered product of Ignimbrite.	
7–8	8.00	98				7.5 7.7	D	35,50/R5cm	50			
8–9	9.00	98				9 9.12	D	50/R12cm	50	12.00	Very stiff, pale white, silty SAND (SM), product of decomposed Ignimbrite.	
9–10	10.00	98				10.5 10.62	D	50/R12cm	50			
10–11	11.00	98				12 12.13	D	50/R13cm	50	16.00	very stiff,dark brown, sandy SILT with gravels (ML), highly weathered product of Ignimbrite.	
11–12	12.00	98				13.5 13.63	D	50/R13cm	50			
12–13	13.00	98				15 15.11	D	50/R11cm	50			
13–14	14.00	98				16.5 16.63	D	50/R13cm	50	20.00	Very stiff, light brown, sandy lean CLAY with gravel (CL), highly weathered product of Ignimbrite.	
14–15	15.00	98										
15–16	16.00	98										
16–17	17.00	98				18 18.1	D	50/R10cm	50			
17–18	18.00	98										
18–19	19.00	98										
19–20	20.00	98										

SPT graph N-value: 4, 24, 44

Driller: Abebe Balcha Logged by: Hagos K/Mariam Checked by: Kassahun Zergaw

Note:
D=Disturbed Sample (Split spoon)
UD=Undisturbed Sample
RC=Rock Core Sample
SPT=Standard Penetration Test

SCR=Solid Core Recovery
RQD=Rock Quality Designation
AFS=Average Fracture Spacing

For Station 12+470

Project	Contractor	Sub Contractor	Consultant
Ankober-Awash Arba Junction Road Project; Contract-I: Ankober-Dulecha	Sunshine Construction PLC	CWCE	C-Tech Engineering PLC

Location/Chainage: Km 12+470	Drilling Method: Dry/Wash Boring by single core barrel (0-20m)	
Coordinates (E/N): 564453.993 1056543.804	Core diameter (mm): 90mm(0-10.0m), 75 mm (10.0 -20m)	Sheet: 1 of 1
Ground Elevation (masl.): 2273.704	Casing type & depth: 110mm	BH Inclination: 90°
Borehole No.: 12+470	Date Started: 26-Apr-2018	Bit type: Carbide/Diamond
Total Depth (m): 20	Date Completed: 1-May-2018	Type of Rig: Wire line drilling

Depth (m)	Core run	TCR (%)	SCR (%)	RQD (%)	AFS (cm)	Sample depth	Sample type	SPT	N	Soil/Rock symbole	Soil/Rock description	Ground water level (m)
0												
1	1.00	98								1.00	Firm,dark grey, top fat clay (CH)	
2	2.00	98				2 2.2	UD					Nil
3	3.00	98				3 3.45	D	5,4,3	7			
4	4.00	98										
5	5.00	98				4.5 4.95	D	7,7,5	12		Firm to very stiff, light brown, lean CLAY with GRAVEL (CL) & Ignimbritic boulders (1-2m)	
6	6.00	98				6 6.45	D	3,4,7	11			
7	7.00	98										
8	8.00	98				7.5 7.95	D	5,12,12	24	8.50		
9	9.00	98				9 9.45	D	10,11,14	25			
10	10.00	98										
11	11.00	98				11 11.4	UD					
12	12.00	98										
13	13.00	98				12.5 12.95	D	9,10,13	23		Very stiff, dark grey, fat CLAY (CH), Highly weathred Ignimbrite & Boulders (16.7-17.5m)	
14	14.00	98				14 14.45	D	10,11,15	25			
15	15.00	98										
16	16.00	98				15.5 15.95	D	13,14,18	32			
17	17.00	98										
18	18.00	98										
19	19.00	98				19 19.45	D	11,13,13	26			
20	20.00									20.00		

SPT graph N-value: 6, 16, 26

Driller: Tarekegn Tesfaye

Logged by: Hagos K/Mariam

Checked by: Kassahun Zergaw

Note:
D=Disturbed Sample (Split spoon)
UD=Undisturbed Sample
RC=Rock Core Sample
SPT=Standard Penetration Test

SCR=Solid Core Recovery
RQD=Rock Quality Designation
AFS=Average Fracture Spacing

For Station 12+490

Project	Contractor	Sub Contractor	Consultant
Ankober-Awash Arba Junction Road Project; Contract-I: Ankober-Dulecha	Sunshine Construction PLC	CWCE	C-Tech Engineering PLC

Location/Chainage: Km 12+490	Drilling Method: Dry/Wash Boring by single core barrel (0-20m)	
Coordinates (E/N): 584474.2 105654.6	Core diameter (mm): 90mm(0-9.0m), 75 mm (9.0 -20m)	Sheet: 1 of 1
Ground Elevation (masl.): 2272.3	Casing type & depth: 110mm	BH Inclination: 90°
Borehole No.: 12+490	Date Started: 21-Apr-2018	Bit type: Carbide/Diamond
Total Depth (m): 20	Date Completed: 26-Apr-2018	Type of Rig: Wire line drilling

Depth (m)	Core run	TCR (%)	SCR (%)	RQD (%)	AFS (cm)	Sample depth	Sample type	SPT	N	Soil/Rock description	Ground water level (m)
0–1	1.00	98								Firm, dark grey, fat CLAY (CH) with some gravels.	
1–2	2.00	98				1.5 1.95	D	4,3,4	7		Nil
2–3	3.00	98				3 3.45	D	11,7,6	13		
3–4	4.00	98				3.8 4	UD				
4–5	5.00	98				4.5 4.8	D	4,50R15cm	50	Firm to very stiff, light brown, lean CLAY (CL)with gravel & some boulders.	
5–6	6.00	98									
6–7	7.00	98				6 6.45	D	8,18,20	38		
7–8	8.00	98									
8–9	9.00	98				8.5 8.95	D	20,24,31	55		
9–10	10.00	98				10 10.45	D	21,24,27	51	Very stiff, light grey, fat CLAY (CH) with some Gravels.	
10–11	11.00	98				11 11.4	UD				
11–12	12.00	98									
12–13	13.00	98				12.5 12.65	D	50R10cm	50		
13–14	14.00	98									
14–15	15.00	98									
15–16	16.00	98								Very stiff, light brown to yellow, sandy lean CLAY (CL) with gravels & weatherd product of IGNIMBRITE bouldrs.	
16–17	17.00	98				16.5 16.7	D				
17–18	18.00	98									
18–19	19.00	98				18.5 18.6	D	50R10cm	50		
19–20	20.00	98									

SPT graph N-value: 4, 24, 44

Soil/Rock symbole: 1.50, 8.00, 12.50, 20.00

Driller: Tarekegn Tesfaye

Logged by: Hagos K/Mariam

Checked by: Kassahun Zergaw

Note:

D=Disturbed Sample (Split spoon)
UD=Undisturbed Sample
RC=Rock Core Sample
SPT=Standard Penetration Test

SCR=Solid Core Recovery
RQD=Rock Quality Designation
AFS=Average Fracture Spacing

APPENDIX–E

Core Box Photograph

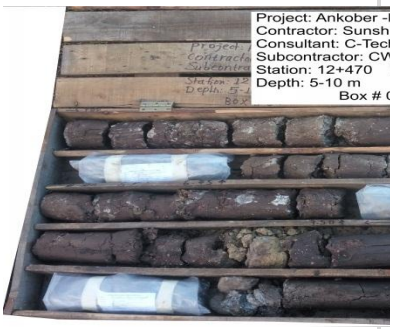
For Station 2+550

Depth (m)	Core box photographs
0-5	
5-10	

For Station 7+130 For Station 7+210

Depth (m)	Core box photographs	Depth (m)	Core box photographs
0-5		0-5	
5-10		5-10	
10-15		10-15	
15-20		15-20	

For Station 12+470 For Station 12+490

Depth (m)	Core box photographs	Depth (m)	Core box photographs
0-5		0-5	
5-10		5-10	
10-15		10-15	
15-20		15-20	